Geotechnical Design Services

New Mental Health Building/Seismic Retrofit of Nursing Tower/New Parking Garage
VA Puget Sound Healthcare System
Seattle, Washington

for
U.S. Department of Veterans Affairs

September 30, 2010

GeoEngineers
Earth Science + Technology

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File No. 9851-006-00

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GEOENGINEERS

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INTRODUCTION

This report presents the results of GeoEngineers' geotechnical engineering services for the proposed new Building 101 Mental Health Services and Research building, new 1000-car parking garage, and seismic retrofit of Building 100 Nursing Tower and the Community Living Center (CLC) at the U.S. Department of Veterans Affairs (VA) Puget Sound Healthcare System facility in Seattle, Washington. The project site is shown relative to surrounding physical features on the Vicinity Map (Figure 1) and the Site Plan (Figure 2).

GeoEngineers submitted drafts of this report dated October 30, 2009 and September 23, 2010 to the project team for review and comment. Comments from the project team and changes to the project have been incorporated into this final report.

GeoEngineers' geotechnical engineering services for this project consisted of:

- Reviewing existing subsurface information available for the site;
- Completing subsurface explorations;
- Conducting design-level studies; and
- Providing geotechnical engineering conclusions and recommendations for the design and construction of the planned Mental Health Services and Research building and parking garage and seismic retrofit of the existing nursing tower.

Additionally, geotechnical seismic design parameters are required by Degenkolb Engineers (Degenkolb), the project structural engineer, for input into their structural models used in the retrofit design of the existing nursing tower and CLC building per American Society of Civil Engineers (ASCE) 41-06 code. In order to complete their design analysis using the nonlinear static procedure, Degenkolb requires the site-specific response spectra for the BSE-1 and BSE-2 earthquake levels that have risk levels of 10 and 2 percent probability of exceedance (PE) in 50 years, respectively.

In addition, seven representative orthogonal sets of earthquake time histories will be selected and scaled in general accordance with the requirements of the ASCE 41-06 code for use in the nonlinear dynamic structural analysis by Degenkolb as a check of the design completed using the nonlinear static procedure.

GeoEngineers' geotechnical engineering services were completed in general accordance with our services agreement dated October 16, 2009.

PROJECT DESCRIPTION

The project consists of three major components: (1) seismic retrofit of Building 100 Nursing Tower and the CLC; (2) design of a new multi-story Mental Health Services and Research building; and (3) design of a new 1000-car parking structure.

The Nursing Tower and CLC structure were built in 1981. The Nursing Tower is an eight-story steel frame structure with composite concrete floors. Each floor has an 8-foot-high interstitial space for mechanical and electrical ducts and conduits. The floor-to-floor height is 18 feet, with the floor-to-interstitial-space height of 10 feet. The CLC is a one-story building with a partial daylight basement and approximate overall dimensions of 200 by 230 feet. The CLC is located adjacent to the Nursing Tower. The seismic retrofit of these structures will consist of correcting the structural deficiencies in order to meet a performance objective of immediate occupancy after a design level earthquake.

The new Mental Health Services and Research building is currently planned to be located in the northwest portion of the VA Seattle campus in an area currently used as a surface parking lot. Construction of the Mental Health Services and Research building will require removal of the existing asphalt concrete pavement parking area and construction of a new multi-story medical and research building. The parking garage is currently planned to be located in the existing surface parking area to the west and north of the existing hospital buildings and will have 3 below grade levels and 4 above grade levels.

Foundation support for the planned buildings will consist of shallow foundations extending to undisturbed glacially consolidated soils or on structural fill extending to undisturbed glacially consolidated soils. The lowest floor levels will be completed as slab-on-grade.

FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

The subsurface soil and groundwater conditions at the site were evaluated by drilling sixteen borings (GEI-1 through GEI-16) in September, 2009 and September 2010. The borings were completed using trailer-mounted, continuous-flight, hollow-stem auger drilling equipment.

The approximate locations of the explorations completed for this project are presented on the Site Plan, Figure 2. Details of the field exploration program and logs of the explorations are presented in Appendix A.

Laboratory Testing

Soil samples were obtained during the drilling program and taken to GeoEngineers' laboratory for further evaluation. Selected samples were tested for the determination of moisture content, percent fines, gradation analyses, and Atterberg limits (plasticity characteristics). The tests were performed in general accordance with test methods of the American Society for Testing and Materials (ASTM). A description of the laboratory testing and the test results are presented in Appendix B.

Geophysical Measurements

Downhole geophysical measurements were completed in a blank casing installed in boring GEI-7 for measurement of the shear wave velocity. The results of the measurements are presented in Appendix C.

PREVIOUS STUDIES

In addition to the explorations completed as part of this study, the logs of explorations completed as part of previous studies in the project vicinity were reviewed. The approximate locations of explorations completed as part of previous studies are also shown in Figure 2, and the exploration logs are included in Appendix D. The existing geotechnical information includes:

- The logs of 29 borings completed on the VA campus by Shannon and Wilson, Inc. in 1979; and
- The logs of two borings completed by Otto Rosenau & Associates in 2008.

SITE CONDITIONS

Geologic Setting

The site is located on an upland glacial drift plain east of Elliot Bay and the Duwamish River valley, and west of Lake Washington. The site is underlain by glacially consolidated soils forming a relatively thin veneer covering the underlying bedrock (Liesch et al., 1963; Troost et al., 2004). The glacially consolidated soils were deposited in the area during the last glacial period that ended approximately 15,000 years ago, during which deep erosion formed Lake Washington and Puget Sound.

The site is located about 1 mile south of the Seattle Fault. The Seattle Fault is a reverse thrust fault, and the site is situated on the upthrusted, or hanging wall, side of the fault. Bedrock is present at the ground surface and at relatively shallow depths on the south side of the Seattle Fault. To the north of the fault, the bedrock basement is several thousand feet below the surface and is overlain by both glacial and nonglacial deposits.

Regional Seismicity

Earthquake Source Zones

The Seattle area is located near the convergent continental boundary known as the Cascadia Subduction Zone (CSZ), an approximately 650-mile-long thrust fault that extends along the Pacific Coast from mid-Vancouver Island to Northern California. The CSZ is the zone where the westward advancing North American Plate is overriding the subducting Juan de Fuca Plate. The interaction of these two plates results in two potential seismic source zones: (1) the Benioff source zone, and (2) the CSZ interplate source zone. A third seismic source zone, referred to as the shallow crustal source zone, is associated with the north-south compression resulting from northerly movement of the Sierra Nevada block of the North American Plate.

BENIOFF SOURCE ZONE

Benioff source zone earthquakes are also referred to as intraplate, intraslab or deep subcrustal earthquakes. Benioff zone earthquakes occur within the subducting Juan de Fuca Plate between depths of 20 and 50 miles and typically have no large aftershocks. Extensive faulting results as the Juan de Fuca Plate is forced below the North American Plate and into the upper mantle.

The Olympia 1949 (M = 7.1), the Seattle 1965 (M = 6.5) and the Nisqually 2001 (M = 6.8) earthquakes are considered to be Benioff zone earthquakes. The Benioff zone is characterized as being capable of generating earthquakes up to magnitude 7.5. The recurrence interval for large earthquakes originating from the Benioff source zone is believed to be shorter than for the shallow crustal and CSZ source zones; damaging Benioff zone earthquakes in Western Washington occur every 30 years or so. The deep focal depth of these earthquakes tends to dampen the shaking intensity when compared to shallow crustal earthquakes of similar magnitudes.

CSZ INTERPLATE SOURCE ZONE

CSZ interplate earthquakes result from rupture of all or a portion of the convergent boundary between the subducting Juan de Fuca Plate and the overriding North American Plate. The fault surfaces approximately 50 to 75 miles off the Washington coast. The width of the seismogenic portion of the CSZ interplate fault varies along its length. As the fault becomes deeper, materials being faulted become ductile, and the fault is unable to store mechanical stresses.

The CSZ is considered to be capable of generating earthquakes of magnitude 8 to magnitude 9 and higher. No earthquakes on the CSZ have been instrumentally recorded; however, through the geologic record and historical records of tsunamis in Japan, it is believed that the most recent CSZ event occurred in the year 1700 (Atwater, 1996; Satake et al., 1996). Recurrence intervals for CSZ interplate earthquakes are thought to be on the order of 400 to 600 years. Paleogeologic evidence suggests that five to seven interplate earthquakes may have been generated along the CSZ over the last 3,500 years at irregular intervals.

SHALLOW CRUSTAL SOURCE ZONE

The shallow crustal source zone is used to characterize shallow crustal earthquake activity within the North American Plate. Shallow crustal earthquakes typically occur at depths ranging from 3 to 20 miles. The shallow crustal source zone is characterized as being capable of generating earthquakes up to about magnitude 7.5. Large shallow crustal earthquakes are typically followed by a sequence of aftershocks.

The largest known earthquakes associated with the shallow crustal source zone in Western Washington include an event on the Seattle Fault about A.D. 900 and the 1872 North Cascades earthquake. The Seattle Fault event was believed to have been magnitude 7 or greater (Johnson et al., 1999), and the 1872 North Cascades earthquake is estimated to have been between magnitudes 6.8 and 7.4. The location of the 1872 North Cascades earthquake is uncertain; however, recent research suggests that the earthquake’s intensity center was near the south end of Lake Chelan (Bakun et al., 2002).

As noted above, the project site is located about 1 mile south of the Seattle Fault zone. The Seattle Fault zone is a 2- to 4-mile-wide, east-west trending zone of three or more splays of the south dipping reverse fault (Johnson et al., 1999). The Seattle Fault ruptured about 1,100 years ago and caused broad uplift and subsidence on both sides of the fault. The rate of recurrence of large earthquakes on the Seattle Fault is thought to be on the order of thousands of years.

Surface Conditions

The VA Puget Sound Healthcare System facility is situated on an approximately 51-acre campus in Seattle’s Beacon Hill neighborhood. The campus is bounded by the Jefferson Park golf club to the north, Asa Mercer Junior High school to the west, Beacon Avenue South to the east and portions of South Snoqualmie Street, South Columbian Way and South Alaska Street to the south. The site is currently occupied by asphalt concrete surface parking lots that border the east, west and south sides of the hospital buildings. The VA Puget Sound Healthcare System campus is composed of approximately 25 buildings that are located in the central and northeast portions of the site. Numerous buried utilities are located within or near the site; these utilities include gas, power, communications, sanitary sewer, storm drain and water.

In general, the site topography slopes down to the south and west. The existing ground surface varies from about Elevation 345 at the north/northeast portion of the property to about Elevation 300 feet along the south side of the property, along South Columbian Way.

Vegetation is limited to trees, shrubs and lawn areas associated with the campus landscaping that surrounds buildings and parking areas. No surface water features were observed in the immediate site vicinity.

Subsurface Conditions

In general, the soils observed in the explorations completed at the project site consisted of fill, glacially consolidated deposits and bedrock. This section describes the units in the order of deposition, starting with the most recent.

Fill consisting of very loose to very dense silty sand with variable gravel content and stiff silt with variable sand content was observed in borings GEI-1, GEI-2, GEI-3, GEI-4, GEI-7, GEI-8, and GEI-9 through GEI-16. Where encountered, the fill extended from the ground surface to approximate depths ranging from less than 1-foot to 17 feet below the ground surface. Borings GEI-5, GEI-5, GEI-6, and GEI-9 through GEI-16 encountered approximately 1½ to 3 inches of asphalt concrete pavement at the ground surface. In some of the boring locations the asphalt concrete pavement was underlain with a base course consisting of fine to coarse gravel with sand with a thickness up to 6 inches thick.

Glacially consolidated soils were encountered at the ground surface at the boring GEI-5 and GEI-6 locations and below the fill in each of the remaining borings completed as part of this study. The glacially consolidated soils encountered in the explorations consist of interbedded layers of medium dense to very dense silty sand with variable gravel content, medium dense to very dense sand with silt and variable gravel content, and very stiff to hard silt and clay of variable plasticity and with variable sand content. Occasional cobbles were inferred from the drilling action. Glacially consolidated soils in the site vicinity are known to contain cobbles and boulders and should be anticipated during construction.

Siltstone and claystone bedrock was encountered in borings GEI-1, GEI-3, GEI-4 and GEI-7 below the glacially consolidated soils and at depths ranging from approximately 27 to 50 feet below existing grades. Bedrock was encountered in many of the explorations completed for previous studies at depths ranging from approximately 24 to 47 feet below site grades.

Groundwater Conditions

Perched groundwater was noted at variable depths on the previous exploration logs. The extent of fine-grained (silt and clay) and silty sand soils noted on the exploration logs indicates that groundwater, where encountered, will likely be associated with isolated zones of perched groundwater throughout the depth of the planned excavations. Groundwater conditions will likely vary by location and season.

CONCLUSIONS AND RECOMMENDATIONS

Summary

A summary of the primary geotechnical considerations is provided below. The summary is presented for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report.

- A site-specific probabilistic site response analysis was completed to assess the BSE-1 and BSE-2 seismic hazard at the site and to develop site-specific response spectra in accordance with the ASCE 41-06 code. These site-specific response spectra should be used for the seismic retrofit of the Nursing Tower and the CLC building. The new parking garage and Mental Health Services and Research building can be designed using the site-specific spectrum or using the 2006 International Building Code (IBC) spectrum for Site Class C.
- Competent glacially consolidated soils are present at relatively shallow depths across the site. The planned parking garage is anticipated to be supported on shallow spread or mat foundations bearing on the native glacially consolidated soils. The Mental Health Services and Research building will be supported on shallow foundations bearing either directly on native glacially consolidated soils or on structural fill/controlled density fill (CDF) extending down to the native glacially consolidated soils. Existing fill within the footprint of the Mental Health Services and Research building should be removed prior to construction of the new building.
- Temporary shoring will be required to construct the below-grade portion of the planned parking garage. Soldier pile walls (with tiebacks where necessary) are recommended for this project due to the variable thickness of fill present at the site, perched groundwater conditions, and system reliability.
- Permanent drainage measures should be incorporated into the design of below-grade walls and below slabs-on-grade, as is standard practice in the Seattle area for excavations into low-permeability soils.
- The lowest level of the parking garage and Mental Health Services and Research building can be constructed as slab-on-grade. Existing fill soils should be removed from the footprint of the Mental Health Services and Research building and replaced with properly compacted structural fill, where new fill is required.
- The soil layering/gradation at the VA Seattle campus is highly heterogeneous. Soil gradation changes significantly with depth in each boring and between adjacent borings. Also, much of the near surface soils have a relatively high fines content and as a result, have lower permeabilities. Due to the variability of the soil gradation and the lower permeability of the near surface soils, infiltration of storm water will be difficult and is not recommended.

Our specific geotechnical recommendations are presented in the following sections of this report.

Seismic Hazards

Ground Rupture

Because of the anticipated infrequent recurrence of earthquake events and the project site's location with respect to the nearest known fault (Seattle Fault), it is our opinion that the risk of ground rupture at the site resulting from surface faulting is low.

Liquefaction Potential

Liquefaction refers to a condition in which vibration or shaking of the ground, usually from earthquake forces, results in development of high excess pore water pressures in saturated soils and subsequent loss of stiffness and/or strength in the deposit of soil so affected. In general, soils that are susceptible to liquefaction include loose to medium dense, clean to silty sands that are below the water table. We conclude that the medium dense to very dense/hard glacially consolidated deposits below the site result in a low potential for liquefaction and liquefaction-induced displacements at the site.

2006 IBC Seismic Design Information

We recommend the 2006 IBC parameters for site class, short period spectral response acceleration (S_S), 1-second period spectral response acceleration (S_1), and seismic coefficients F_A and F_V presented in the following table. These values are based on the 2002 United States Geological Survey (USGS) Seismic Hazard Maps.

2009 IBC Parameter	Recommended Value
Site Class	C
Short Period Spectral Response Acceleration, S_S (percent g)	155
1-Second Period Spectral Response Acceleration, S_1 (percent g)	53
Seismic Coefficient, F_A	1.0
Seismic Coefficient, F_V	1.30

Site-Specific Response Spectra

A site-specific seismic hazard analysis per ASCE 41-06 Section 1.6.2 was completed to develop the response spectra for the BSE-1 and BSE-2 earthquake levels for use in the seismic retrofit of the Nursing Tower and the CLC building. The recommended site-specific horizontal and vertical response spectra are presented in Figures 3 and 4, respectively.

The response spectra at the site was evaluated using the published ground motion prediction equations (GMPEs) (or attenuation relations) and by completing probabilistic 1-dimensional site-specific seismic response analysis. Site response spectra were calculated at two foundation depths: one at the ground surface and the other at 15 feet below the ground surface. The site-specific response spectra were developed by probabilistically integrating the results of the published GMPEs and the results of the probabilistic seismic response analysis.

The vertical response spectra were developed by first calculating the ratio of vertical to horizontal (V/H) response spectra for period range of 0 to 3 seconds using the GMPEs developed by Abrahamson and Silva (1997) and by Campbell and Bozorgnia (2003). The resulting V/H ratios were then integrated probabilistically with the horizontal response spectra to compute the site-specific vertical response spectra.

The seismic hazard calculation and probabilistic site response analysis were completed by Dr. Walter J. Silva with Pacific Engineering and Analysis (PE&A) as a subconsultant to GeoEngineers. Details of the site-specific seismic hazard analysis are presented in Appendix E.

Earthquake Time Histories

Seven representative orthogonal sets of earthquake time histories will be used by Degenkolb as input for the nonlinear dynamic analysis. These seven sets of earthquake time histories need to be scaled such that the average of the square root of the sum of squares (SRSS) spectra from all horizontal component pairs does not fall below 1.3 times the corresponding ordinate of the site-specific response spectrum within the period range of interest, as required by ASCE 41-06 Section 1.6.2.2. Based on the analysis completed by Degenkolb, the period range of interest is approximately 1 to 1.5 seconds for the Nursing Tower and is approximately 0.1 to 0.3 seconds for the CLC.

Based on the results of the seismic hazard deaggregation analyses by USGS, the seismic hazard at the project site is dominated by Seattle Fault for periods between 0.1 and 2 seconds. The seven earthquakes presented in the table below were selected to be representative of the seismic hazard for this project. Five of the selected records represent the Seattle Fault earthquake hazard, one represents the Benioff earthquake hazard and one represents the CSZ interplate earthquake hazard. The plots of acceleration versus time for each of the unscaled motions are presented in Figures 5 through 11.

Earthquake	M	Station	Distance (km)	Recorded Peak Ground Acceleration (PGA)
Landers 1992	7.3	Lucerne	1	0.79 (NS)/0.73 (EW)/0.82 (UP)
Iran 1978	7.4	Tabas	3	0.84 (NS)/0.85 (EW)/0.69 (UP)
Loma Prieta 1989	7.0	Gilroy	34	0.33 (NS)/0.36 (EW)/0.19 (UP)
San Fernando 1971	6.6	Pacoima Kagel Canyon	18	1.23 (NS)/1.16 (EW)/0.70 (UP)
Taiwan Chi-Chi 1999	7.6	TCU071	5	0.65 (NS)/0.57 (EW)/0.45 (UP)
Nisqually 2001	6.8	Alki	73	0.04 (NS)/0.02 (EW)/0.02 (UP)
Mexico-Michoacan 1985	8.1	Villita	48	0.10 (NS)/0.11 (EW)/0.06 (UP)

The table below presents the scaling factors developed for the horizontal (H) and vertical (V) components of each of the earthquake time histories for the Nursing Tower and CLC building for the BSE-1 and BSE-2 earthquake levels.

Earthquake	Nursing Tower		CLC Building	
	BSE-1	BSE-2	BSE-1	BSE-2
Landers 1992	0.3 (H), 0.5 (V)	0.8 (H), 1.0 (V)	0.5 (H), 0.7 (V)	1.0 (H), 1.4 (V)
Iran 1978	0.3 (H), 0.4 (V)	0.6 (H), 0.7 (V)	0.5 (H), 0.5 (V)	0.7 (H), 1.0 (V)
Loma Prieta 1989	0.9 (H), 2.0 (V)	2.3 (H), 4.0 (V)	1.2 (H), 1.6 (V)	2.3 (H), 3.2 (V)
San Fernando 1971	0.2 (H), 0.5 (V)	0.6 (H), 1.0 (V)	0.4 (H), 0.4 (V)	0.6 (H), 0.8 (V)
Taiwan Chi-Chi 1999	0.4 (H), 0.5 (V)	0.8 (H), 1.0 (V)	0.7 (H), 0.7 (V)	1.0 (H), 1.4 (V)
Nisqually 2001	3.2 (H), 4.0 (V)	8.0 (H), 8.0 (V)	5.0 (H), 5.5 (V)	8.0 (H), 11.0 (V)
Mexico-Michoacan 1985	1.6 (H), 2.0 (V)	4.0 (H), 4.0 (V)	2.0 (H), 2.8 (V)	4.0 (H), 5.6 (V)

The average SRSS spectra of the horizontal components of the seven scaled earthquake time histories is presented in Figures 12 and 13 for the Nursing Tower and CLC building, respectively. For comparison purposes, Figures 12 and 13 also show 1.3 times the recommended response spectra for both the BSE-1 and BSE-2 earthquake levels. Figures 14 and 15 show the average vertical spectra of the seven scaled earthquake time histories for the Nursing Tower and CLC building, respectively.

We understand that Degenkolb will include the soil-foundation interaction effect in the structural analysis via the use of soil springs. The foundation soil springs will be developed using the methodologies specified in ASCE 41-06 Section C4.4.2 that require the effective shear modulus of the foundation soils as input. The effective shear modulus of the soils can be correlated with the effective shear wave velocity of the foundation soils. Based on the shear wave velocity measurement and the shear modulus reduction factor per ASCE 41-06, the effective shear wave velocity for the foundation soil near the ground surface is recommended to be 570 feet per second (ft/s). For the foundation soil below a depth of 15 feet, the recommended effective shear wave velocity is 750 ft/s.

Excavation Support

The appropriate temporary shoring system depends on subsurface soil and groundwater conditions, excavation depth, deflection tolerances and proximity of existing structures. Temporary shoring will be required for the planned parking garage excavation and we understand that the temporary shoring will be designed by the contractor. Conventional soldier pile and tieback shoring is recommended for the planned parking garage due to: 1) the thickness of fill soils at the site, 2) the presence of perched groundwater, 3) the need to protect existing improvements and limit vertical and horizontal deformations that could damage existing improvements, and 4) soldier pile and tieback shoring is anticipated to be a more reliable shoring system. General design recommendations for temporary soldier pile and tieback walls are presented below to assist the project team during the design phase.

Excavation Considerations

The site soils may be excavated with conventional excavation equipment, such as trackhoes or dozers. It may be necessary to rip the glacially consolidated soils locally to facilitate excavation. The contractor should be prepared for occasional cobbles and boulders in the site soils.

Likewise, the surficial fill may contain foundation elements and/or utilities from previous site development, debris, rubble and/or cobbles and boulders. We recommend that procedures be identified in the project specifications for measurement and payment of work associated with obstructions.

Soldier Pile and Tieback Walls

Soldier pile walls consist of steel beams that are concreted into drilled vertical holes located along the wall alignment, typically 8 feet on center. After excavation to specified elevations, tiebacks are installed, if necessary. Once the tiebacks are installed, the pullout capacity of each tieback is tested, and the tieback is locked-off to the soldier pile at or near the design tieback load. Tiebacks typically consist of steel strands or bars that are installed into pre-drilled holes and then either tremie or pressure grouted. Timber lagging is typically installed behind the flanges of the steel beams to retain the soil located between the soldier piles. Geotechnical design recommendations for each of these components of the soldier pile and tieback wall system are presented in the following sections.

SOLDIER PILES

We recommend that soldier pile walls be designed using the earth pressure diagrams presented in Figures 16 and 17. The earth pressures presented in Figure 16 are for full height cantilever soldier pile walls. The earth pressures presented in Figure 17 are for full-height soldier pile walls with a single level or multiple levels of tiebacks. Recommended surcharge pressures for design of the shoring walls are presented in Figure 18. The earth pressures presented in Figures 16 through 18 represent the estimated loads that will be applied to the wall system for various wall heights.

The earth pressures presented in Figures 16 and 17 include the loading from traffic surcharge. Additional surcharge loads (floor or foundation loads, etc.) can be evaluated using the surcharge pressures presented in Figure 18. Other surcharge loads, such as cranes, construction equipment or construction staging areas, should be considered by GeoEngineers on a case-by-case basis. In Figures 16 and 17, no seismic pressures have been included because it is assumed that the shoring will be temporary.

We recommend that the embedded portion of the soldier piles be at least 2 feet in diameter and extend a minimum distance of 10 feet below the base of the excavation to resist “kick-out.” The axial capacity of the soldier piles must resist the downward component of the anchor loads and other vertical loads, as appropriate. We recommend using an allowable end bearing value of 40 kips per square foot (ksf) for piles supported on the glacially consolidated soils. The allowable end bearing value should be applied to the base area of the drilled hole into which the soldier pile is concreted. This value includes a factor of safety of about 2.5. The allowable end bearing value assumes that the shaft bottom is cleaned out immediately prior to concrete placement. If necessary, an allowable pile skin friction of 1.5 ksf may be used on the embedded portion of the soldier piles to resist the vertical loads.

LAGGING

We recommend that the temporary timber lagging be sized using the procedures outlined in the Federal Highway Administration’s Geotechnical Engineering Circular No. 4. The site soils are best described as competent soils. The following table presents recommend timber lagging thicknesses (roughcut) as a function of soldier pile clear span and depth. Shotcrete lagging can be used as an alternative to timber lagging, depending upon the contractor's preference.

Depth (feet)	Recommended Lagging Thickness (roughcut) for clear spans of:					
	5 feet	6 feet	7 feet	8 feet	9 feet	10 feet
0 to 25	2 inches	3 inches	3 inches	3 inches	4 inches	4 inches
25 to 50	3 inches	3 inches	3 inches	4 inches	4 inches	5 inches

Lagging should be installed promptly after excavation, especially in areas where perched groundwater is present or where clean sand and gravel soils are present and caving soils conditions are likely. The workmanship associated with lagging installation is important for maintaining the integrity of the excavation.

The space behind the lagging should be filled with soil as soon as practicable. The voids behind the lagging should be backfilled immediately or within a single shift, depending on the selected method of backfill. Placement of backfill will help reduce the risk of voids developing behind the wall and damage to existing improvements located behind the wall.

Lean concrete is a suitable option for the use of backfill behind the walls. Lean concrete will reduce the volume of voids present behind the wall. Alternatively, lean concrete may be used for backfill behind the upper 15 to 20 feet of the excavation to limit caving and sloughing of the upper soils, with on-site soils used to backfill the voids for the remainder of the excavation. Based on our experience, the voids between each lean concrete lift are sufficient for preventing the buildup of hydrostatic pressure behind the wall.

TIEBACKS

Tieback anchors can be used for wall heights where cantilever soldier pile walls are not cost-effective. Tieback anchors should extend far enough behind the wall to develop anchorage beyond the "no-load" zone and within a stable soil mass. The anchors should be inclined downward at 15 to 25 degrees below the horizontal. Corrosion protection will not be required for the temporary tiebacks with a design life of less than one year.

Centralizers should be used to keep the tieback in the center of the hole during grouting. Structural grout or concrete should be used to fill the bond zone of the tiebacks. A bond breaker, such as plastic sheathing, should be placed around the portion of the tieback located within the no-load zone if the shoring contractor plans to grout both the bond zone and unbonded zone of the tiebacks in a single stage. If the shoring contractor does not plan to use a bond breaker to isolate the no-load zone, GeoEngineers should be contacted to provide recommendations.

Loose soil and slough should be removed from the holes drilled for tieback anchors prior to installing the tieback. The contractor should take necessary precautions to minimize loss of ground and prevent disturbance to previously installed anchors and existing improvements in the site vicinity. Holes drilled for tiebacks should be grouted/filled promptly to reduce the potential for loss of ground.

Tieback anchors should develop anchorage in the glacially consolidated soils. We recommend that spacing between tiebacks be at least 3 times the diameter of the anchor hole to minimize group interaction. We recommend a preliminary design load transfer value between the anchor and soil of 3 kips/foot for glacially consolidated soils and 1.5 kips/foot for fill soil. Higher adhesion values may be developed, depending on the anchor installation technique. The contractor should be given the opportunity to use higher adhesion values by conducting performance tests prior to the start of installing the production tieback anchors.

The tieback anchors should be verification- and proof-tested to confirm that the tiebacks have adequate pullout capacity. The pullout resistance of tiebacks should be designed using a factor of safety of 2. The pullout resistance should be verified by completing at least two successful verification tests in each soil type and a minimum of four total tests for the project. Each tieback should be proof-tested to 133 percent of the design load. Verification and proof tests should be completed as described in Appendix F, "Ground Anchor Load Tests and Shoring Monitoring Program."

The tieback layout and inclination should be checked to confirm that the tiebacks do not interfere with adjacent buried utilities.

DRAINAGE

A suitable drainage system should be installed to prevent the buildup of hydrostatic groundwater pressures behind the soldier pile and lagging wall. It may be necessary to cut weep holes through the lagging in wet areas. Seepage flows at the base of the excavation should be contained and controlled. Drainage should be provided for permanent below-grade walls as described below in the "Below-Grade Walls" section of this report.

CONSTRUCTION CONSIDERATIONS

Temporary casing or drilling fluid may be required to install the soldier piles and possibly the tiebacks where:

- Loose fill is present;
- The native soils do not have adequate cementation or cohesion to prevent caving or raveling;
- Perched groundwater is present.

GeoEngineers should be allowed to observe and document the installation and testing of the shoring to verify conformance with the design assumptions and recommendations.

Temporary Dewatering

The static groundwater table is located below the planned base of the excavation. This conclusion is based on the boring data, our experience with nearby deep excavations and the nature of the soils at the site.

For planning purposes, we recommend that the contractor plan to use sumps and pumps located within the excavation for any required temporary dewatering associated with perched groundwater. For planning purposes, groundwater flow rates of up to 15 gallons per minute can be assumed. Surface water from rainfall will likely contribute significantly to the volume of water that needs to be removed from the excavation during construction and will vary as a function of season and precipitation.

Shallow Foundations

We recommend that the planned buildings be supported on conventional spread or mat foundations bearing on undisturbed glacially consolidated soils or on structural fill/CDF extending down to the undisturbed glacially consolidated soils. Existing fill soils, where present below planned foundation subgrade elevation, should be removed and replaced with properly compacted structural fill. The planned parking garage foundation elevations are anticipated to be within the glacially consolidated soils and little or no fill will be required below foundation elements. The Mental Health Services and Research building foundation elevations vary from below existing grades and within glacially consolidated soils to above existing grades where removal of existing fill and placement of new fill will be required.

As described in Subsurface Conditions above, the site soils consist of a variable thickness of fill overlying the glacially consolidated soils (bearing soils). The estimated contours of the elevation of the fill/glacially consolidated soils contact are presented on the Site Plan, Figure 2. The contours of the fill/glacially consolidated contact presented in Figure 2 represent the estimated highest elevation of the bearing soils across the site. The contours in Figure 2 are based on interpretation of the soil conditions at the location of widely spaced borings completed at the site and represent the best estimate of the top of bearing soils elevation (no conservatism is included in the interpretation). The actual elevation of the bearing soils between the boring locations may vary from the value presented at a specific location. Additionally, because continuous soil sampling was not completed in the borings, the contact between the fill and glacially consolidated soils may be estimated where the contact occurs between soil samples, thus adding uncertainty. It should be recognized that variations in soil conditions and potential differences in actual and recommended bearing elevations may be encountered during construction.

The contours in Figure 2 have been provided for the project team to estimate the amount of earthwork required for construction of the planned buildings. It is recommended that a contingency be included for some over-excavation by the contractor and for potential differences between actual and estimated bearing elevations.

Allowable Bearing Pressure

For foundations constructed as recommended in this report, we recommend using an allowable bearing pressure of 12 ksf for shallow foundations bearing on the dense to very dense glacially consolidated deposits or controlled density fill (CDF) extending down to dense to very dense glacially consolidated deposits. Mat foundations bearing on very dense glacially consolidated soils may be designed using an allowable bearing pressure of 10 ksf. Where 1 to 5 feet of structural fill will be placed below the foundation subgrade elevation, we recommend that the foundations be designed for an allowable bearing pressure of 10 ksf. Where between 5 and 10 feet of structural fill will be placed below the foundation subgrade elevation, we recommend that the foundations be designed for an allowable bearing pressure of 8 ksf. GeoEngineers should be contacted for guidance if more than 10 feet of structural fill will be required below the planned foundation subgrade elevation.

The allowable soil bearing pressures are net values and apply to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads. The allowable soil bearing pressures assume that all loose soil is removed and that the subgrade is prepared using an excavator equipped with a smooth bucket without disturbance of the native soils.

Settlement

Provided all loose soil is removed and the subgrade is prepared as recommended under “Construction Considerations” below, we estimate the total settlement of shallow foundations will be about 1 inch or less. The settlements will occur rapidly, essentially as loads are applied. Differential settlements between footings could be half of the total settlement. Note that smaller settlements will result from lower applied loads.

Lateral Resistance

Lateral foundation loads may be resisted by passive resistance on the sides of footings and by friction on the base of the shallow foundations. For shallow foundations supported on native soils, the allowable frictional resistance may be computed using a coefficient of friction of 0.4 applied to vertical dead-load forces.

The allowable passive resistance may be computed using an equivalent fluid density of 400 pounds per cubic foot (pcf) (triangular distribution). These values are appropriate for foundation elements that are poured directly against undisturbed glacially consolidated soils or surrounded by properly compacted structural fill.

The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

Construction Considerations

If soft areas or existing fill is present below the foundation subgrade elevation, the soft areas/existing fill should be removed and replaced with lean concrete or structural fill. In such instances, the zone of structural fill should extend laterally beyond the footing edges for a horizontal distance at least equal to the thickness of the fill. Where CDF is used to replace fill or soft soils, the CDF should replace the fill/soft soils under the full footprint of the foundation and should extend down to undisturbed native glacially consolidated soils. In such instances, the geotechnical engineer shall verify that the subgrade has been properly prepared.

Glacially consolidated soils are susceptible to softening from water or construction traffic. If necessary, we recommend that the contractor be prepared to pour a mud mat consisting of lean concrete across the exposed foundation subgrade to protect it from softening during wet weather conditions or where groundwater seepage is present.

We recommend that GeoEngineers observe the condition of all subgrade areas to evaluate whether the work is completed in accordance with our recommendations and whether the subsurface conditions are as anticipated.

Slab-on-Grade Floors

Subgrade Preparation

A variable thickness of undocumented fill is present below the footprint of the proposed Mental Health Services and Research building. Also the Mental Health Services and Research building has variable foundation and slab-on-grade elevations. Given the combination of variable undocumented fill thickness and foundation/slab-on-grade elevations, it is recommended that the existing fill be removed from within the Mental Health Services and Research building footprint and that new fill, where required, be placed as structural fill. The estimated elevation of the fill/glacially consolidated soil contact presented on Figure 2 can be used when estimating earthwork quantities.

The exposed slab-on-grade subgrade should be evaluated after site grading is complete. Proof-rolling with heavy, rubber-tired construction equipment should be used for this purpose during dry weather and if access for this equipment is practical. Probing should be used to evaluate the subgrade during periods of wet weather or if access is not feasible for construction equipment. The exposed soil should be firm and unyielding, and without significant groundwater. Disturbed areas should be recompacted if possible or removed and replaced with compacted structural fill. The slabs-on-grade will bear on structural fill or glacially consolidated soils. These soils can be locally soft when wet and may not provide adequate support for construction equipment. A granular work pad over the prepared slab subgrade may facilitate construction activities, depending upon weather conditions, soil conditions and/or the presence of perched groundwater at the subgrade elevation. The work pad will help prevent subgrade disturbance, facilitate construction traffic and aid in removal of rainwater and groundwater seepage. GeoEngineers can work with the contractors to identify whether a work pad is required based on conditions observed during construction, the thickness of the work pad and the work pad material gradation. Typical work pads consist of a 6- to 12-inch-thick layer of clean granular fill such as Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.16. The work pad material should be placed in one lift over the subgrade and be compacted using a smooth drum roller.

Based on our experience with other projects, it is unlikely that it will be possible to prevent the work pad from becoming contaminated with fines during the installation of foundations and/or below-slab utilities such as plumbing. Therefore, it will likely not be possible to use the work pad as the underslab drainage layer. We recommend that the excavation be extended sufficiently below the slab subgrade elevation to accommodate the work pad as well as the underslab capillary break.

An alternative to the use of a work pad is to leave the subgrade elevation approximately 1-foot higher than final grade during foundation construction and installation of below-slab utilities. After completion of this work, the remaining approximately 1-foot of soil will be removed and immediately replaced with the underslab capillary break layer. With this alternative, it still may be necessary to construct temporary access roads for construction traffic, especially during periods of wet weather. The decision to utilize a work pad or to leave the subgrade high until capillary break placement should be made by the general contractor.

Design Parameters

Conventional slabs may be supported on-grade, provided the subgrade soils are prepared as recommended under the "Subgrade Preparation" section above. We recommend that the slab be founded on either undisturbed glacially consolidated soils or on structural fill placed over the undisturbed glacially consolidated soils. For slabs designed as a beam on an elastic foundation, a modulus of subgrade reaction of 250 pounds per cubic inch (pci) may be used for subgrade soils prepared as recommended.

We recommend that the slab-on-grade floors be underlain by a 6-inch-thick capillary break consisting of material meeting the requirements of Mineral Aggregate Type 22 (⅝-inch crushed gravel), City of Seattle Standard Specification 9-03.16, with the exception that this material should have less than 10 percent sand and less than 3 percent fines.

Provided that loose soil is removed and the subgrade is prepared as recommended, we estimate that slabs-on-grade will not settle appreciably.

Below-Slab Drainage

We expect the static groundwater level to be located at or below the slab-on-grade level for the proposed building, and perched groundwater may be present above the slab subgrade elevation. We recommend installing an underslab drainage system to remove water from below the slab-on-grade. The underslab drainage system should include an interior perimeter drain and one longitudinal drain. The drains should consist of perforated Schedule 40 polyvinyl chloride (PVC) pipes with a minimum diameter of 4 inches placed in a trench at least 12 inches deep. The top of the underslab drainage system trenches should coincide with the base of the capillary break layer. The underslab drainage system pipes should have adequate slope to allow positive drainage to the sump/gravity drain.

The drainage pipe should be either machine-slotted or perforated. The slots should be a maximum of ⅛-inch wide with four slots per inch and extend over the lower 60-degree perimeter of the pipe. Perforated pipe should have two rows of ½-inch holes spaced 120 degrees apart and at 4 inches on center. The underslab drainage system trenches should be backfilled with Mineral Aggregate Type 22 or Type 5 (1-inch washed gravel), City of Seattle Standard Specification 9-03.16, or an alternative approved by GeoEngineers. The Type 22 or Type 5 material should be wrapped with a geotextile filter fabric meeting the requirements of construction geotextile for underground drainage, Washington State Department of Transportation (WSDOT) Standard Specification 9-33. The underslab drainage system pipes should be connected to a header pipe and routed to a sump or gravity drain. Appropriate cleanouts for drainpipe maintenance should be installed. A larger-diameter pipe will allow for easier maintenance of drainage systems.

If no special waterproofing measures are taken, leaks and/or seepage may occur in localized areas of the below-grade portion of the building, even if the recommended wall drainage and below-slab drainage provisions are constructed. If leaks or seepage is undesirable, below-grade waterproofing should be specified. A vapor barrier should be used below slab-on-grade floors located in occupied portions of the building.

Below-Grade Walls

Permanent Subsurface Walls

Permanent below-grade walls should be designed for the same earth pressures (including surcharge pressures where applicable) as the adjacent temporary walls, and should also include a seismic load acting over the height of the wall equal to 8H pounds per square foot (psf), where H is the height of the wall in feet. Other surcharge loads, such as from foundations, construction equipment or construction staging areas, should be considered on a case-by-case basis. We can provide the lateral pressures from these surcharge loads as the design progresses.

The soil pressures recommended above assume that wall drains will be installed to prevent the buildup of hydrostatic pressure behind the walls, as described above in the "Excavation Support" section of this report, and tied to permanent drains to remove water to suitable discharge points.

Other Cast-in-Place Walls

Conventional cast-in-place walls may be necessary for small retaining structures located on-site. The lateral soil pressures acting on conventional cast-in-place subsurface walls will depend on the nature, density and configuration of the soil behind the wall and the amount of lateral wall movement that can occur as backfill is placed.

For walls that are free to yield at the top at least 0.1 percent of the height of the wall, soil pressures will be less than if movement is limited by such factors as wall stiffness or bracing. Assuming that the walls are backfilled and drainage is provided as outlined in the following paragraphs, we recommend that yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 35 pcf (triangular distribution), while non-yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 55 pcf (triangular distribution). For seismic loading conditions, a rectangular earth pressure equal to 8H psf should be added to the active/at-rest pressures. Other surcharge loading should be applied as appropriate. Lateral resistance for conventional cast-in-place walls can be provided by frictional resistance along the base of the wall and passive resistance in front of the wall in accordance with the previous "Lateral Resistance" discussion.

The above soil pressures assume that wall drains will be installed to prevent the buildup of hydrostatic pressure behind the walls, as discussed below.

Drainage

Drainage behind the permanent below-grade walls is typically provided using drainage material attached to the lagging of soldier pile shoring walls or located behind the shotcrete facing if used in lieu of timber lagging. The drainage material should be connected to weep pipes that extend through the exterior building wall at the footing elevation. The weep pipes should be connected to perimeter footing drains that are in turn routed to a sump.

The earth pressures presented in Figures 16 and 17 assume that adequate drainage is provided behind the wall. Prefabricated geocomposite drainage material, such as MiraDrain 6000™, should be installed vertically to the face of the lagging. For soldier pile shoring walls, the drainage material should be installed on the excavation side of the timber lagging with the fabric adjacent to the lagging. Where shotcrete facing is used, drainage strips should be installed between the soil and the back of the shotcrete facing.

Full wall face coverage is preferable for minimizing spotting and leaking at the face of the permanent wall. However, the use drainage strips, typically a minimum of 16 inches wide, placed between the piles or behind the shotcrete (if shotcrete facing is used) is sufficient for the structural integrity of the wall. The drainage strips or full wall face coverage should extend the entire height of the wall. If drainage strips are used, additional drainage strips may be necessary in wet areas. Although the use of full wall face coverage will reduce spotting or leaking at the face of the permanent wall, there is still a potential for seepage. If this is a concern, waterproofing should be specified.

Positive drainage should also be provided behind cast-in-place retaining walls by placing a minimum 2-foot-wide zone of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.16, with the exception that the percent passing the U.S. No. 200 sieve is to be less than 3 percent.

A perforated PVC pipe with a minimum diameter of 4 inches should be located at the base of all walls to remove water that collects in this zone. The drainpipe should be placed with 0.5 percent minimum slopes and discharge to an appropriate location with sumps and pumps for discharge.

Earthwork

Structural Fill

Fill placed to support structures, placed behind retaining structures, and placed below pavements and sidewalks will need to be specified as structural fill as described below:

- If structural fill is necessary beneath building footings and slabs, the fill should meet the requirements of Mineral Aggregate Type 2 or Type 17 (1¼-inch minus crushed rock or bank run gravel), City of Seattle Standard Specification 9-03.16.
- Structural fill placed behind retaining walls should meet the requirements of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.16.
- Structural fill placed within utility trenches and below pavement and sidewalk areas should meet the requirements of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.16.
- Structural fill placed as crushed surfacing base course below pavements and sidewalks should meet the requirements of Mineral Aggregate Type 2 (1¼-inch minus crushed rock), City of Seattle Standard Specification 9-03.16.

ON-SITE SOILS

The on-site soils are moisture-sensitive and generally have natural moisture contents higher than the anticipated optimum moisture content for compaction. As a result, the on-site soils will likely require moisture-conditioning in order to meet the required compaction criteria during dry weather conditions and will not be suitable for reuse during wet weather. Furthermore, most of the fill soils required for the project have specific gradation requirements, and the on-site soils do not meet these gradation requirements. Therefore, imported structural fill meeting the requirements listed above should be used where structural fill is necessary.

FILL PLACEMENT AND COMPACTION CRITERIA

Structural fill should be mechanically compacted to a firm, non-yielding condition. Structural fill should be placed in loose lifts not exceeding 1-foot in thickness. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Structural fill should be compacted to the following criteria:

- Structural fill placed in building areas (supporting foundations or slab-on-grade floors) and in pavement and sidewalk areas (including utility trench backfill) should be compacted to at least 95 percent of the maximum dry density (MDD) estimated in accordance with American Society for Testing and Materials (ASTM) D 1557.
- Structural fill placed against subgrade walls should be compacted to between 90 and 92 percent of the MDD. Care should be taken when compacting fill against subsurface walls to avoid overcompaction and hence overstressing the walls.

We recommend that GeoEngineers be present during probing of the exposed subgrade soils in subgrade areas, and during placement of structural fill. We will evaluate the adequacy of the subgrade soils and identify areas needing further work, perform in-place moisture-density tests in the fill to verify compliance with the compaction specifications, and advise on any modifications to the procedures that may be appropriate for the prevailing conditions.

WEATHER CONSIDERATIONS

During wet weather, some of the exposed soils could become muddy and unstable. If so affected, we recommend that:

- The ground surface in and around the work area should be sloped so that surface water is directed to a sump or discharge location. The ground surface should be graded such that areas of ponded water do not develop.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.

Temporary Slopes

Temporary slopes may be used around the site to facilitate early installation of shoring or in the transition between levels at the base of the excavation. We recommend that temporary slopes constructed in the fill be inclined at 1½H:1V (horizontal to vertical) and that temporary slopes constructed in the glacially consolidated soils be inclined at 1H:1V. Flatter slopes may be necessary if seepage is present on the face of the cut slopes or if localized sloughing occurs. For open cuts at the site, we recommend that:

- No traffic, construction equipment, stockpiles or building supplies be allowed at the top of the cut slopes within a distance of at least 5 feet from the top of the cut.
- Exposed soil along the slope are protected from surface erosion using waterproof tarps or plastic sheeting.
- Construction activities are scheduled so that the length of time the temporary cut is left open is reduced to the extent practicable.
- Erosion control measures are implemented as appropriate such that runoff from the site is reduced to the extent practicable.
- Surface water is diverted away from the slope.
- The general condition of the slopes is observed periodically by the geotechnical engineer to confirm adequate stability.

Because the contractor has control of the construction operations, the contractor should be made responsible for the stability of cut slopes, as well as the safety of the excavations. Shoring and temporary slopes must conform to applicable local, state and federal safety regulations.

Recommended Additional Geotechnical Services

GeoEngineers should be retained to review the final project plans and specifications when complete to confirm that our design recommendations have been implemented as intended.

During construction, GeoEngineers should observe the installation of the shoring system, review/collect shoring monitoring data, evaluate the suitability of the foundation subgrades, confirm removal of existing fill soils (where necessary), observe installation of subsurface drainage measures, evaluate structural backfill and provide a summary letter of our construction observation services. The purposes of GeoEngineers' construction phase services are to confirm that the subsurface conditions are consistent with those observed in the explorations and other reasons described in Appendix G, "Report Limitations and Guidelines for Use."

LIMITATIONS

We have prepared this draft final report for the exclusive use of the U.S. Department of Veterans Affairs, Stantec, Coughlin Porter Lundeen, Degenkolb Engineers and their authorized agents for the project site. The data should be provided to prospective contractors for their bidding or estimating purposes, but our report and interpretations should not be construed as a warranty of the subsurface conditions.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

Please refer to Appendix G, "Report Limitations and Guidelines for Use," for additional information pertaining to use of this report.

We appreciate the opportunity to participate on this project. Should you have any questions concerning this report or if we can be of additional service, please call.

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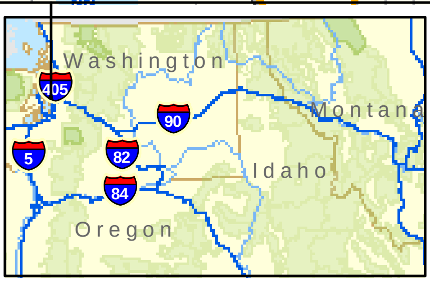
FIGURES



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Feet

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.
3. It is unlawful to copy or reproduce all or any part thereof, whether for personal use or resale, without permission.

Data Sources: ESRI Data & Maps, Street Maps 2005

Transverse Mercator, Zone 10 N North, North American Datum 1983
North arrow oriented to grid north

Vicinity Map

New Mental Health Building / Seismic Retrofit of Nursing Tower / New Parking Garage
Seattle, Washington

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Figure 1

Office: RED Path: P:\9\0985100\6\GIS\0985100600 F1.mxd Map Revised: September 18, 2009 MM2

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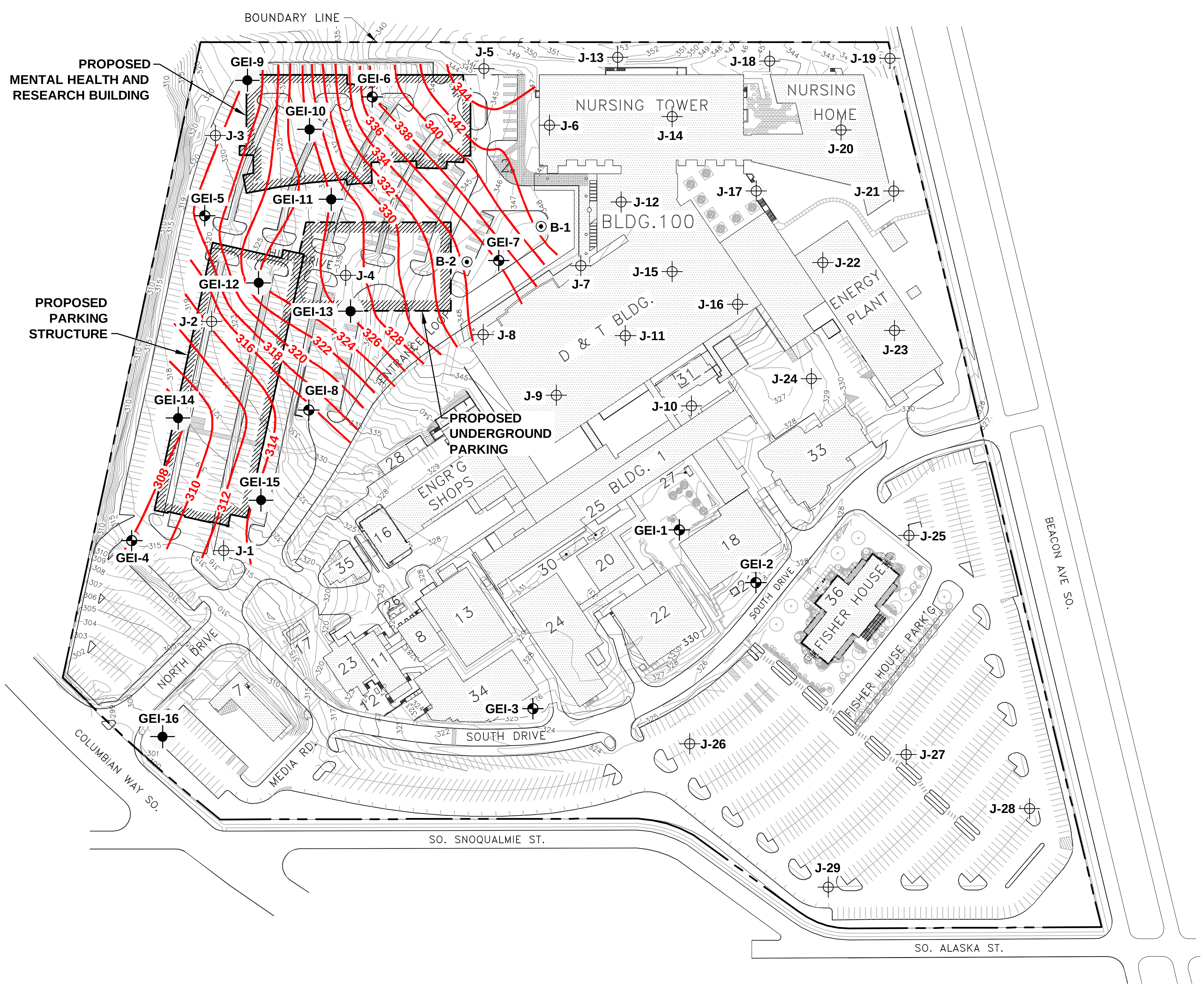

Legend

GEI-9 — Approximate location of boring completed by GeoEngineers, 2010

GEI-1 — Approximate location of boring completed by GeoEngineers, 2009

J-7 — Approximate location of boring completed by Shannon & Wilson, 1979

B-1 — Approximate location of boring completed by Otto Rosenau & Associates, Inc., 2008

320 — Approximate elevation of fill/glacially consolidated soil contact

Notes

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. can not guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Reference: "Site Plan, VAPSHC Seattle Campus", by Construction Management, dated 12/08/08; survey "9050 TOPO" by PLS, Inc., dated 10/15/2009; ground contours "SVA_Survey flat.dwg".

Datum = NAD83

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FEET

Site Plan

New Mental Health Building/Seismic Retrofit of Nursing Tower/New Parking Garage
Seattle, Washington

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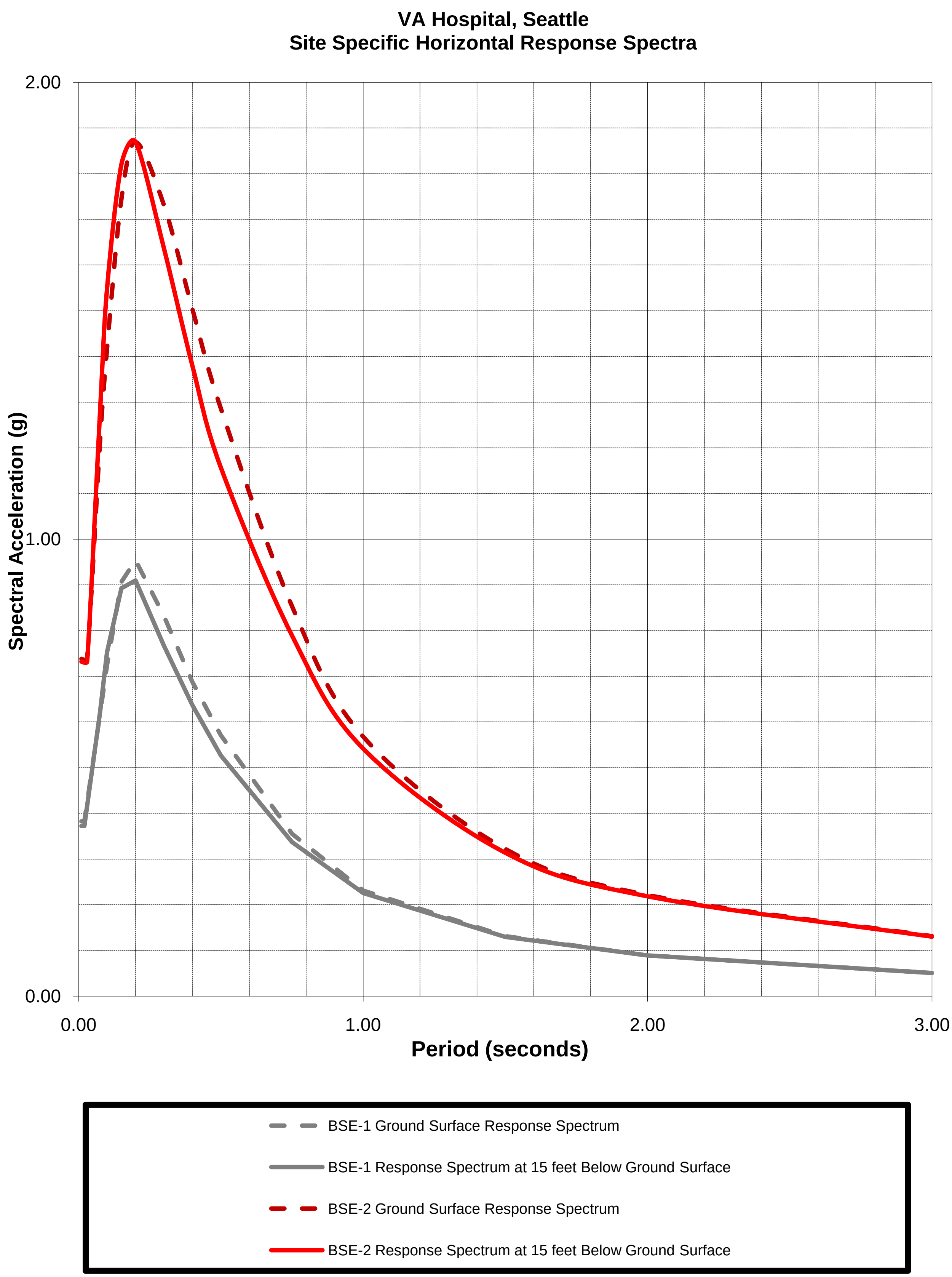
Figure 2



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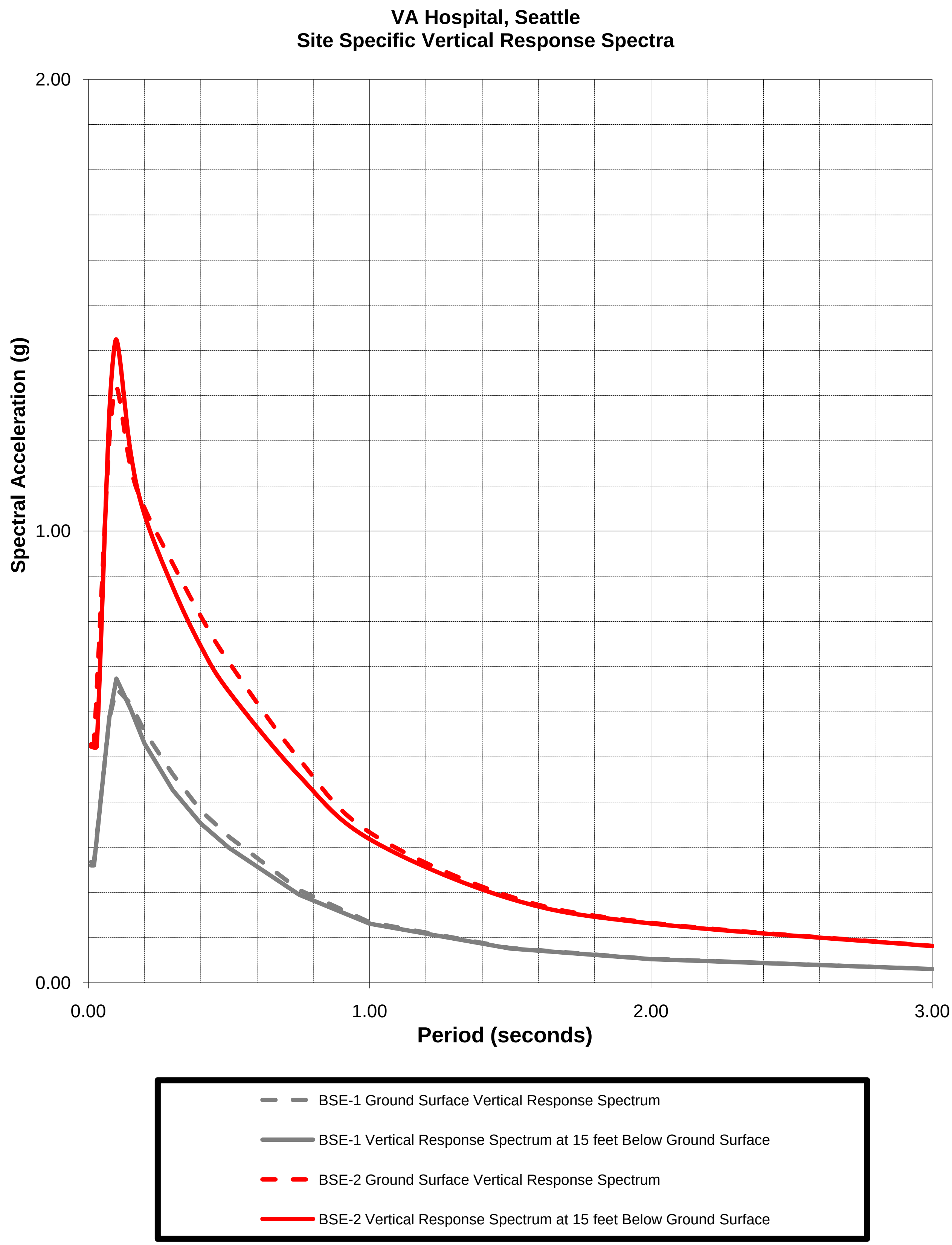
5% Damped Horizontal Response Spectra

Figure 3



9851-006-00 10/28/2009 KHC:Site Specific Response Spectra.xls

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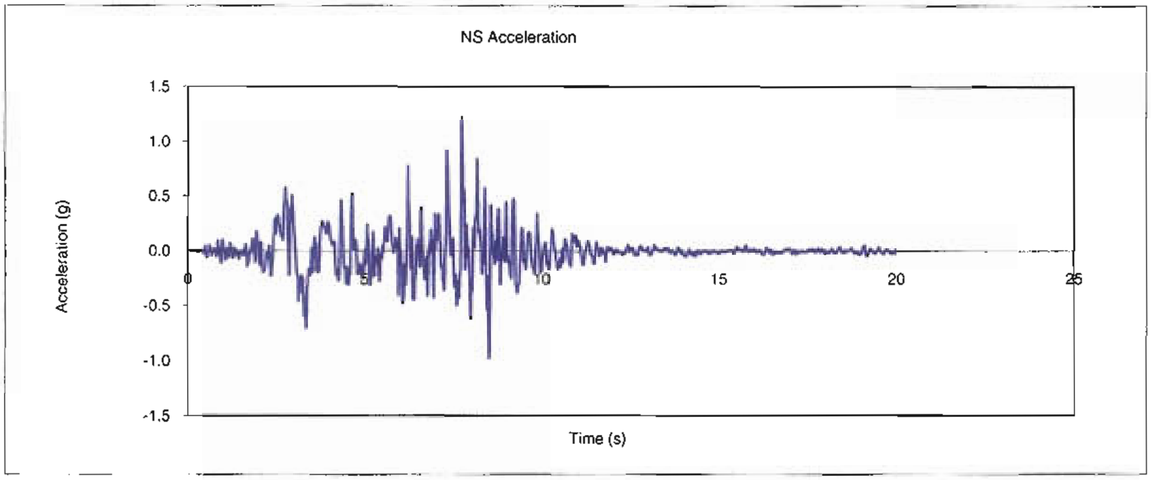
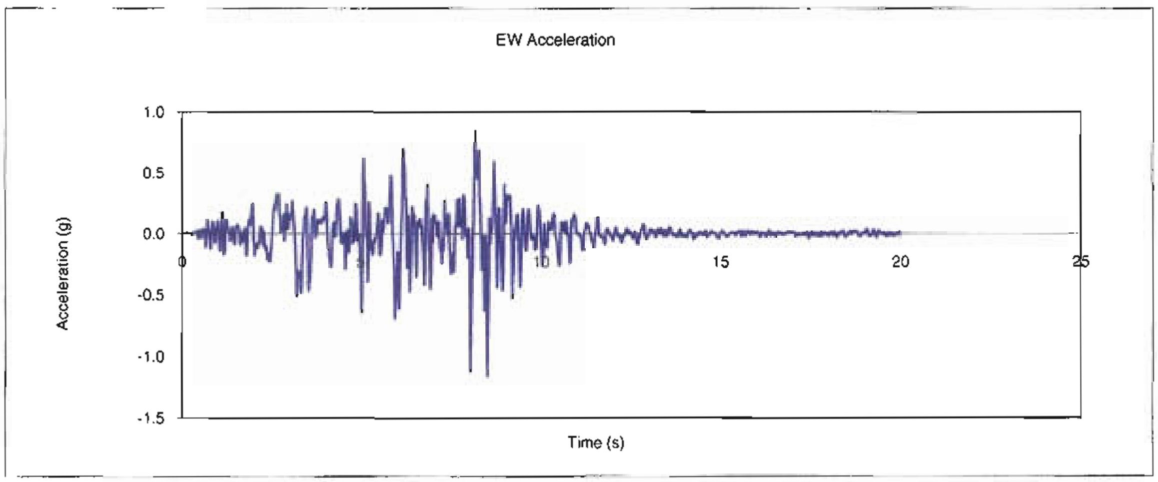
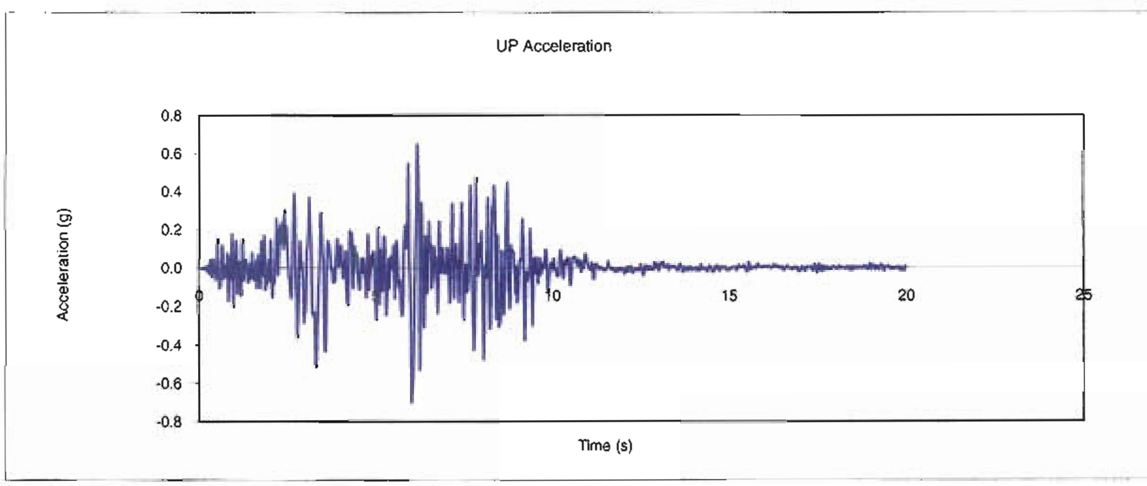
5% Damped Vertical Response Spectra

Figure 4

Earthquake Time Histories







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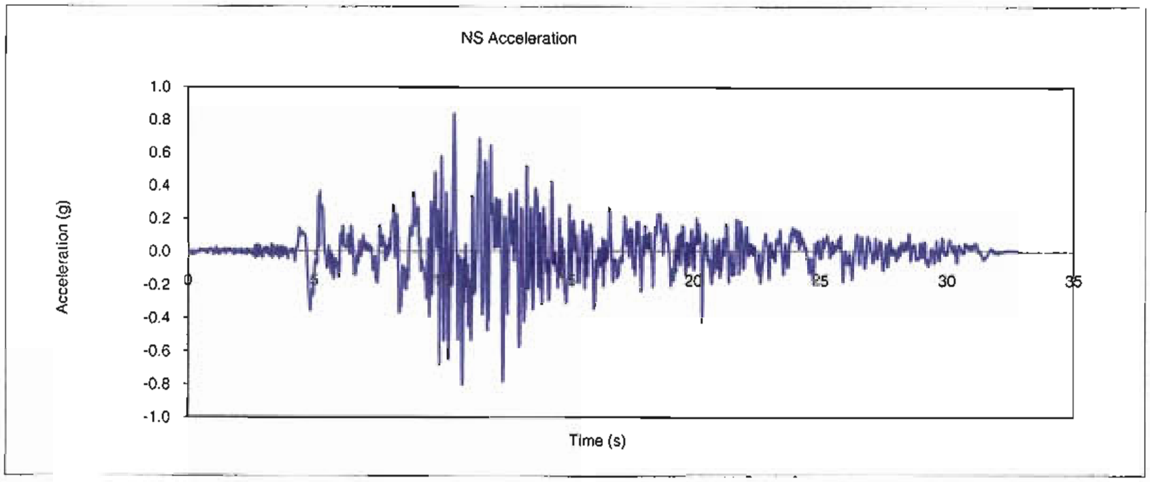
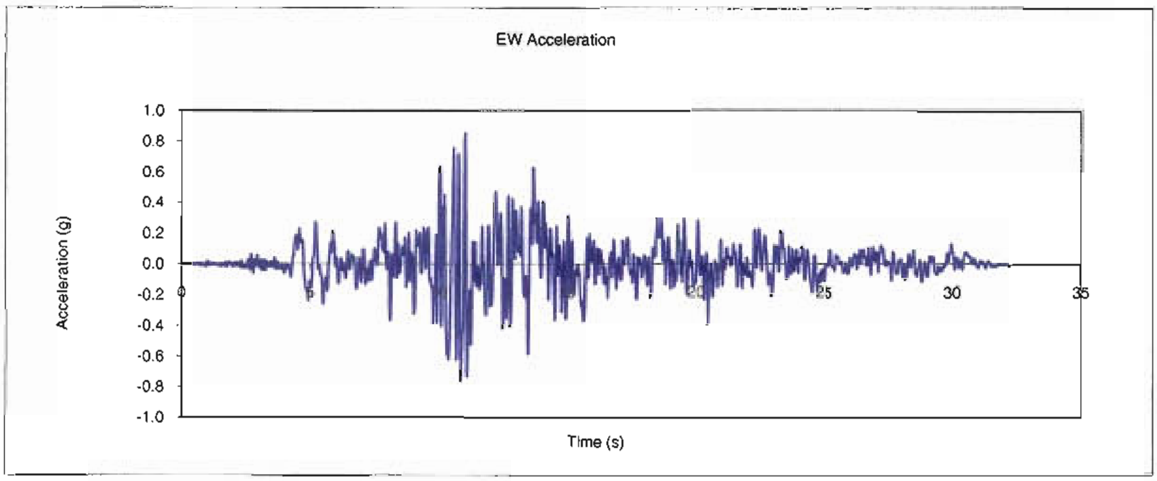
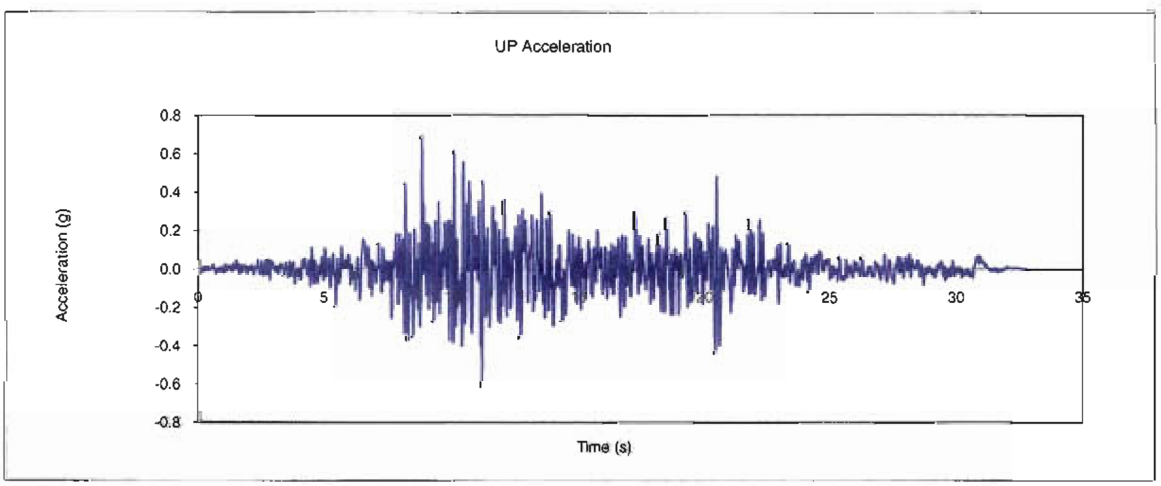
1971 SAN FERNANDO EARTHQUAKE

FIGURE 5

Earthquake Time Histories







GeoEngineers
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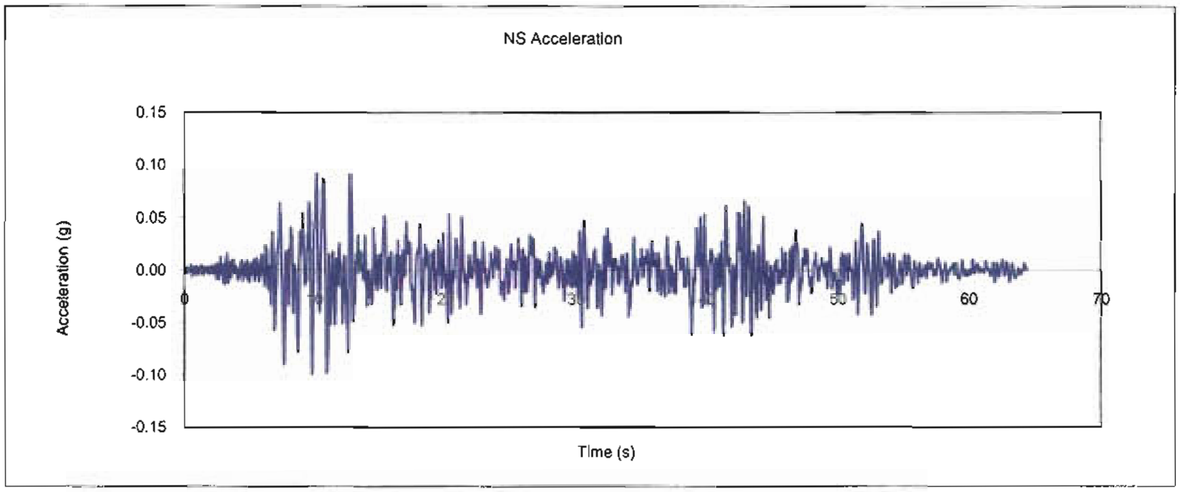
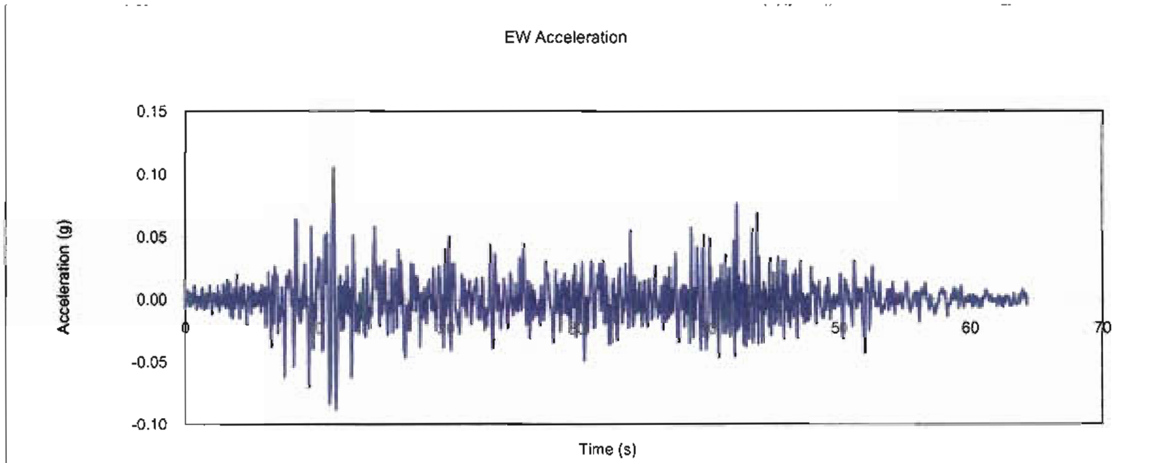
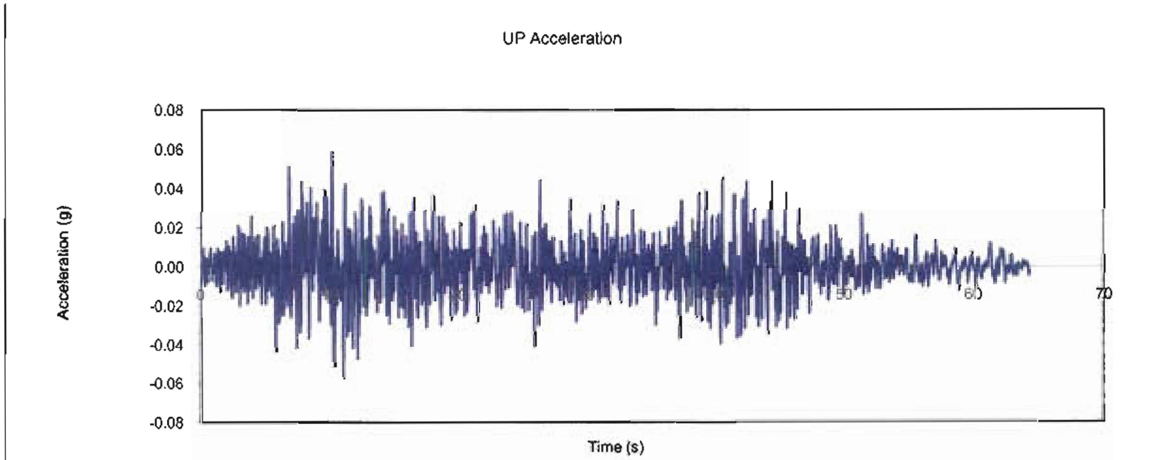
1979 IRAN TABAS EARTHQUAKE

FIGURE 6

Earthquake Time Histories







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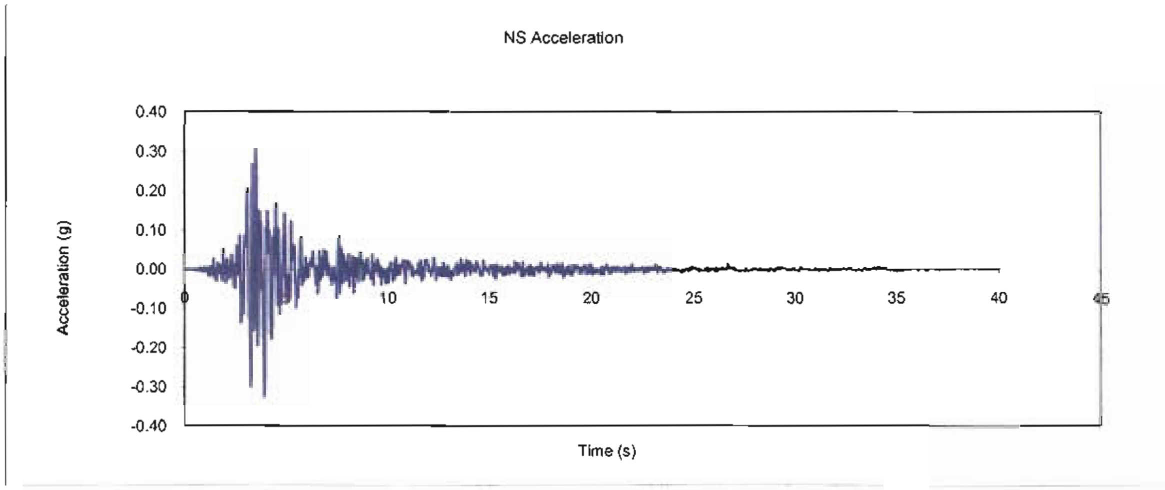
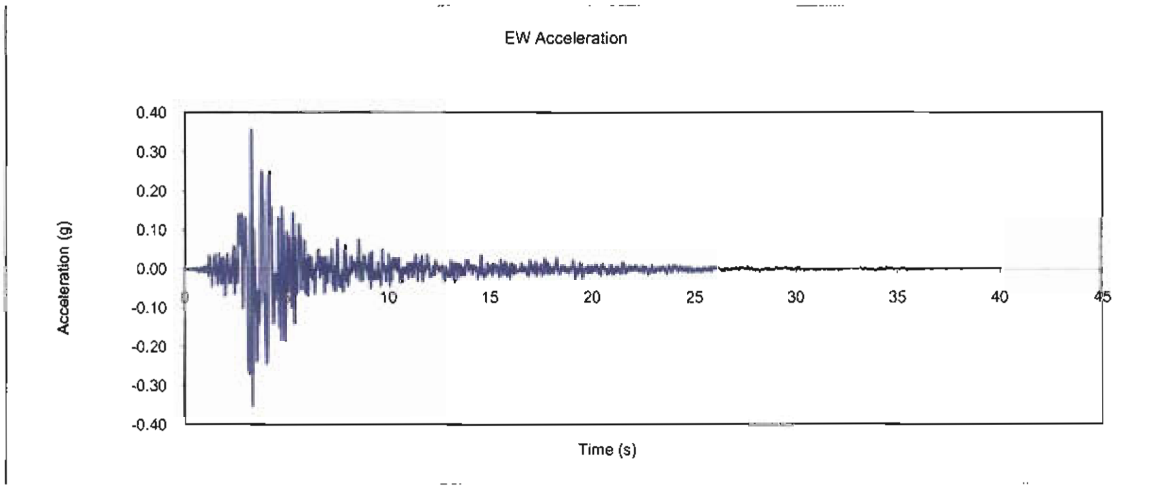
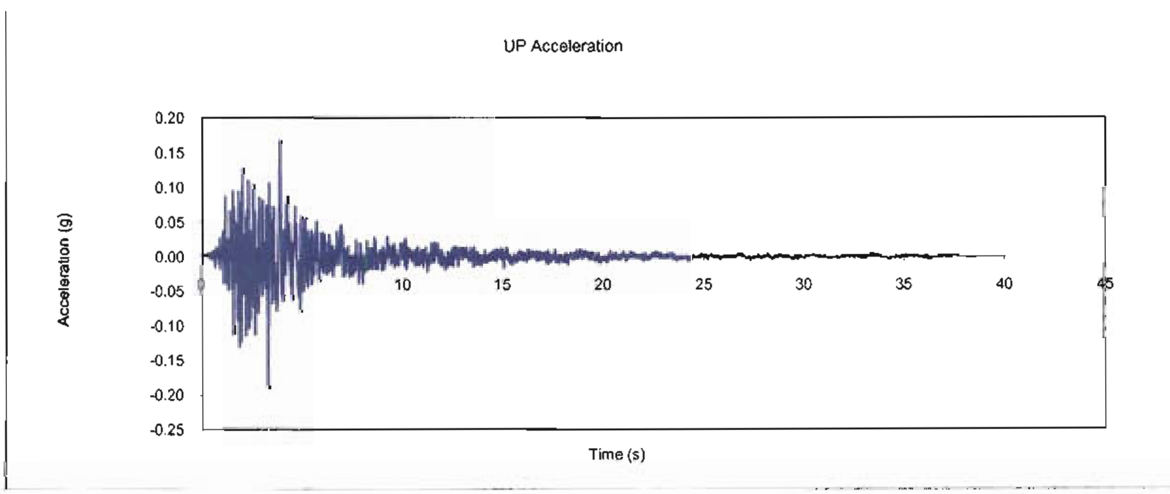
1985 MICHOACAN VILLITA EARTHQUAKE

FIGURE 7

Earthquake Time Histories







GEOENGINEERS
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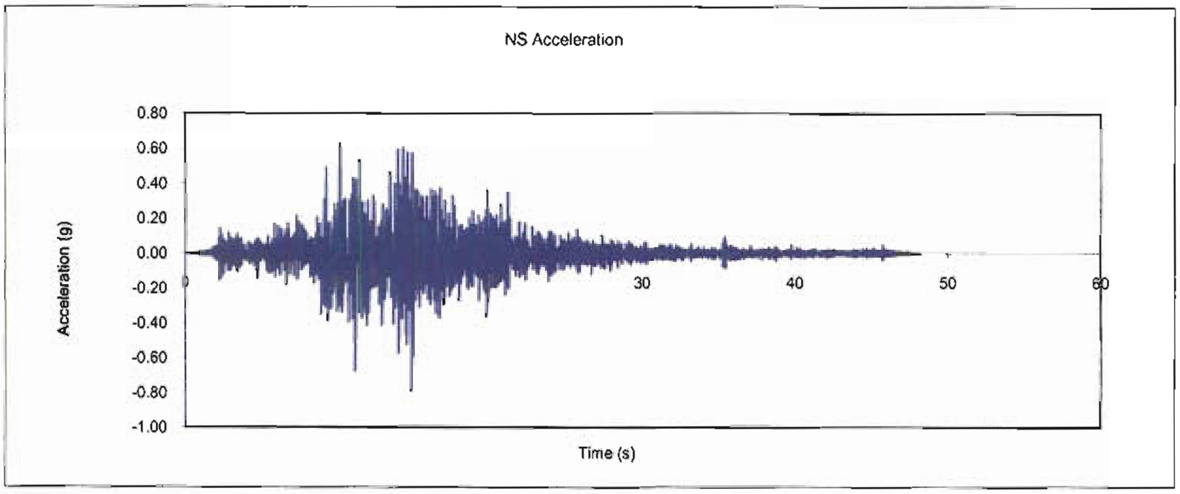
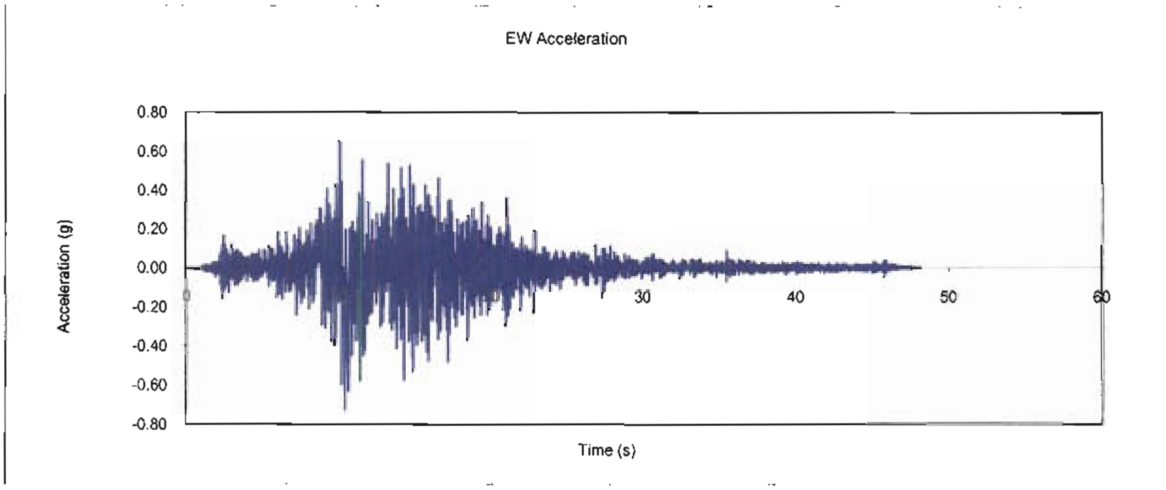
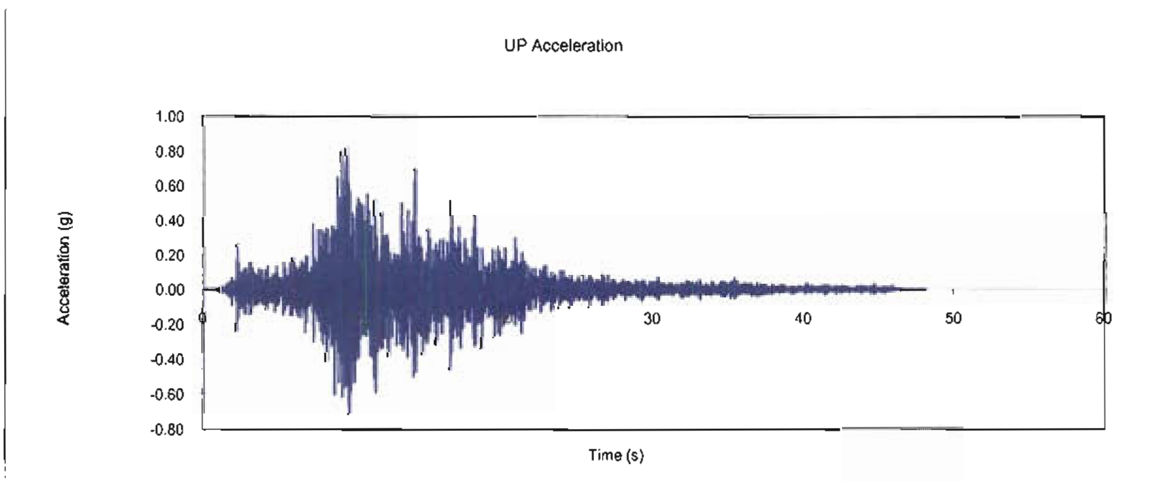
1989 LOMA PRIETA GILROY EARTHQUAKE

FIGURE 8

Earthquake Time Histories







GEOENGINEERS
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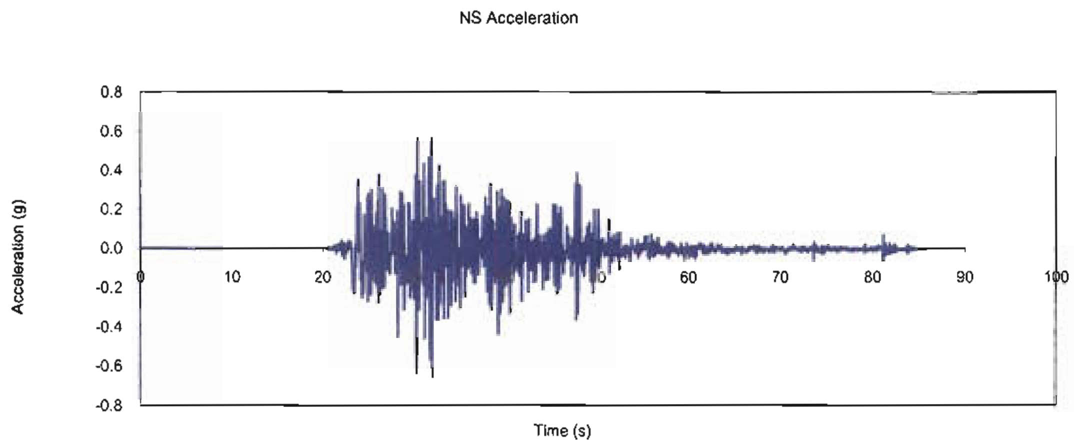
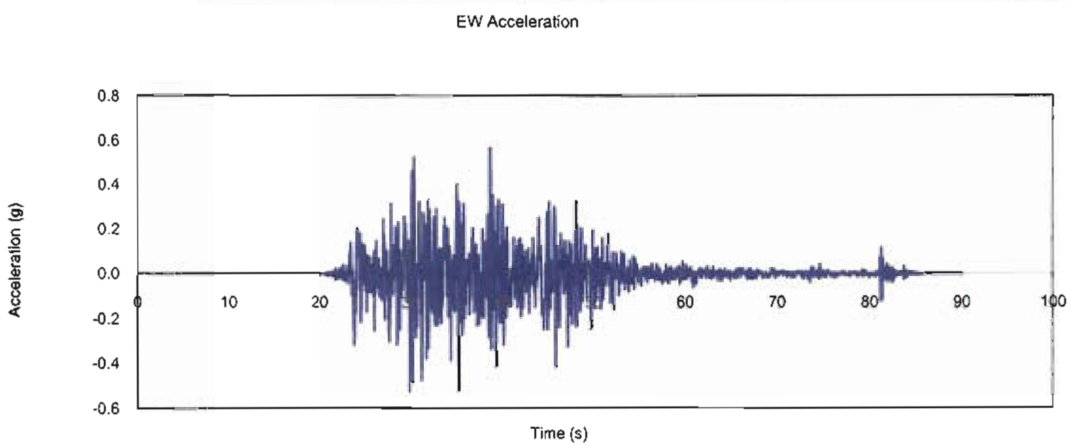
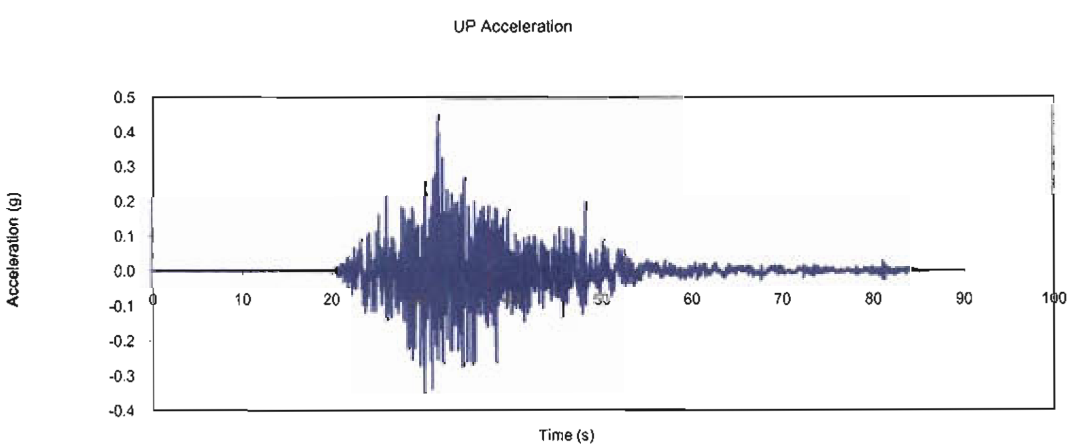
1992 LANDERS LUCERNE EARTHQUAKE

FIGURE 9

Earthquake Time Histories





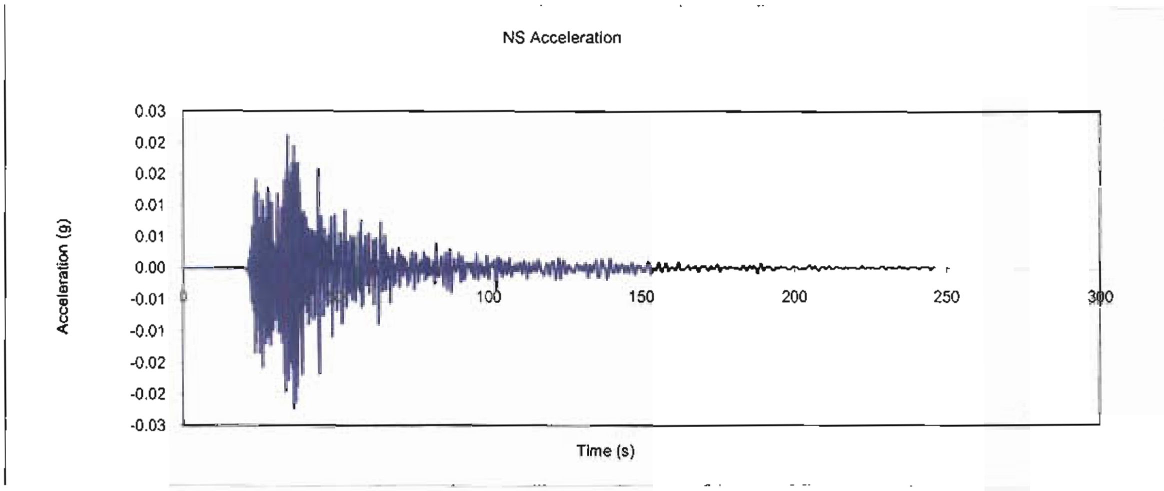


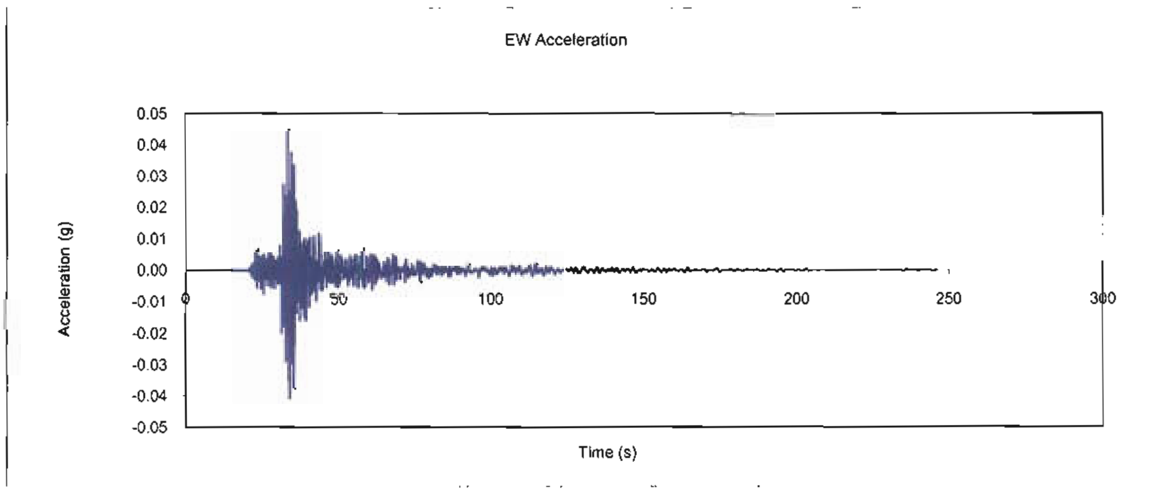
GEOENGINEERS
Earth Science + Technology

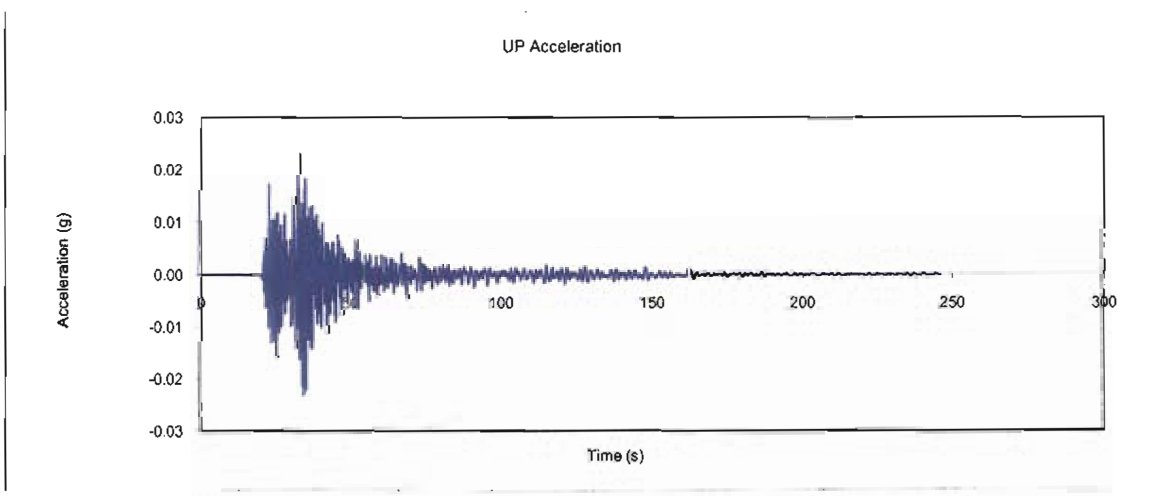
1999 TAIWAN CHI CHI EARTHQUAKE

FIGURE 10

Earthquake Time Histories



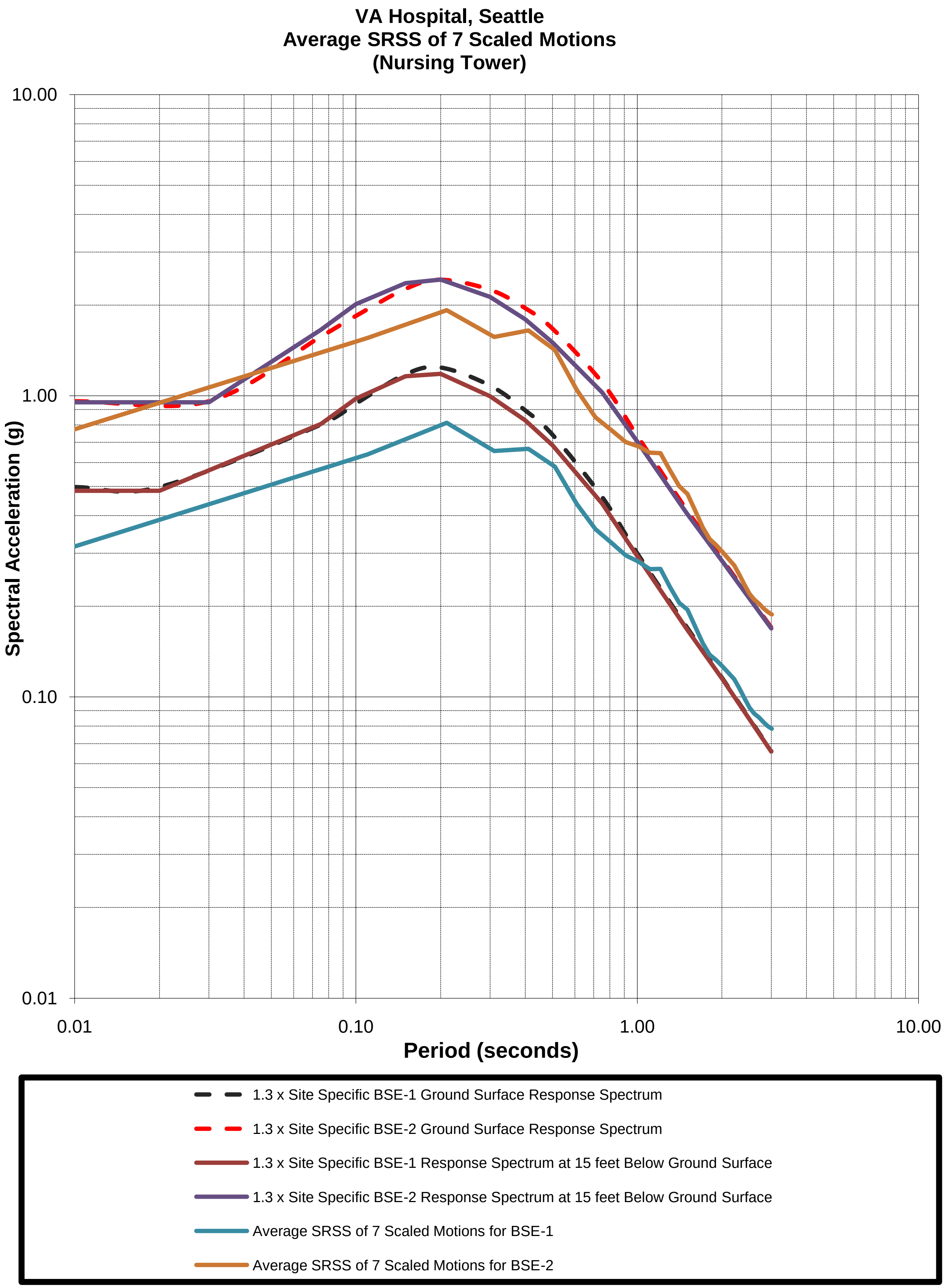




GEOENGINEERS
Earth Science + Technology

2001 NISQUALLY ALKI EARTHQUAKE

FIGURE 11



9851-006-00 10/28/2009 KHC:Site Specific Response Spectra.xls

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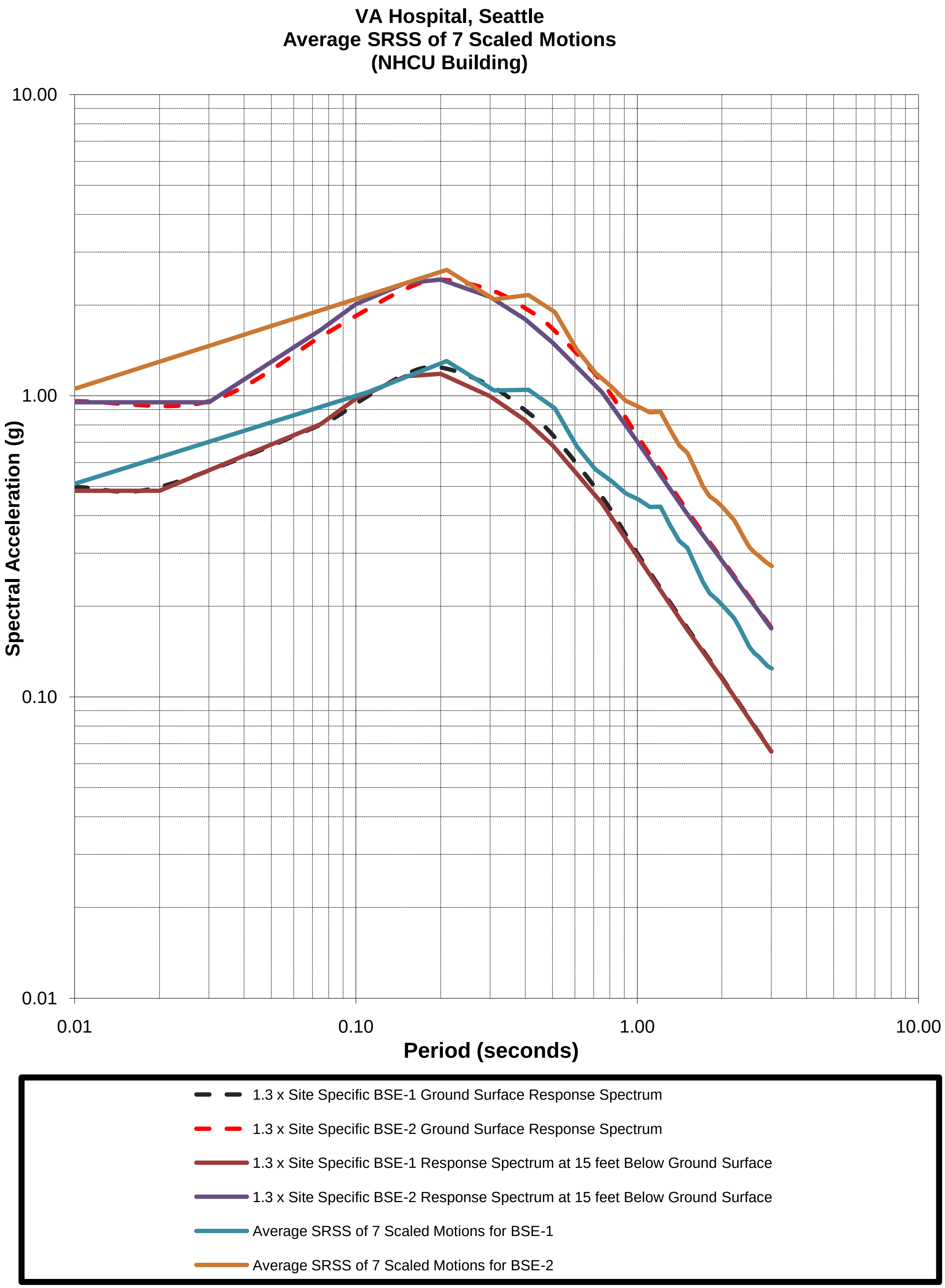
5% Damped Horizontal Response Spectra

Figure 12



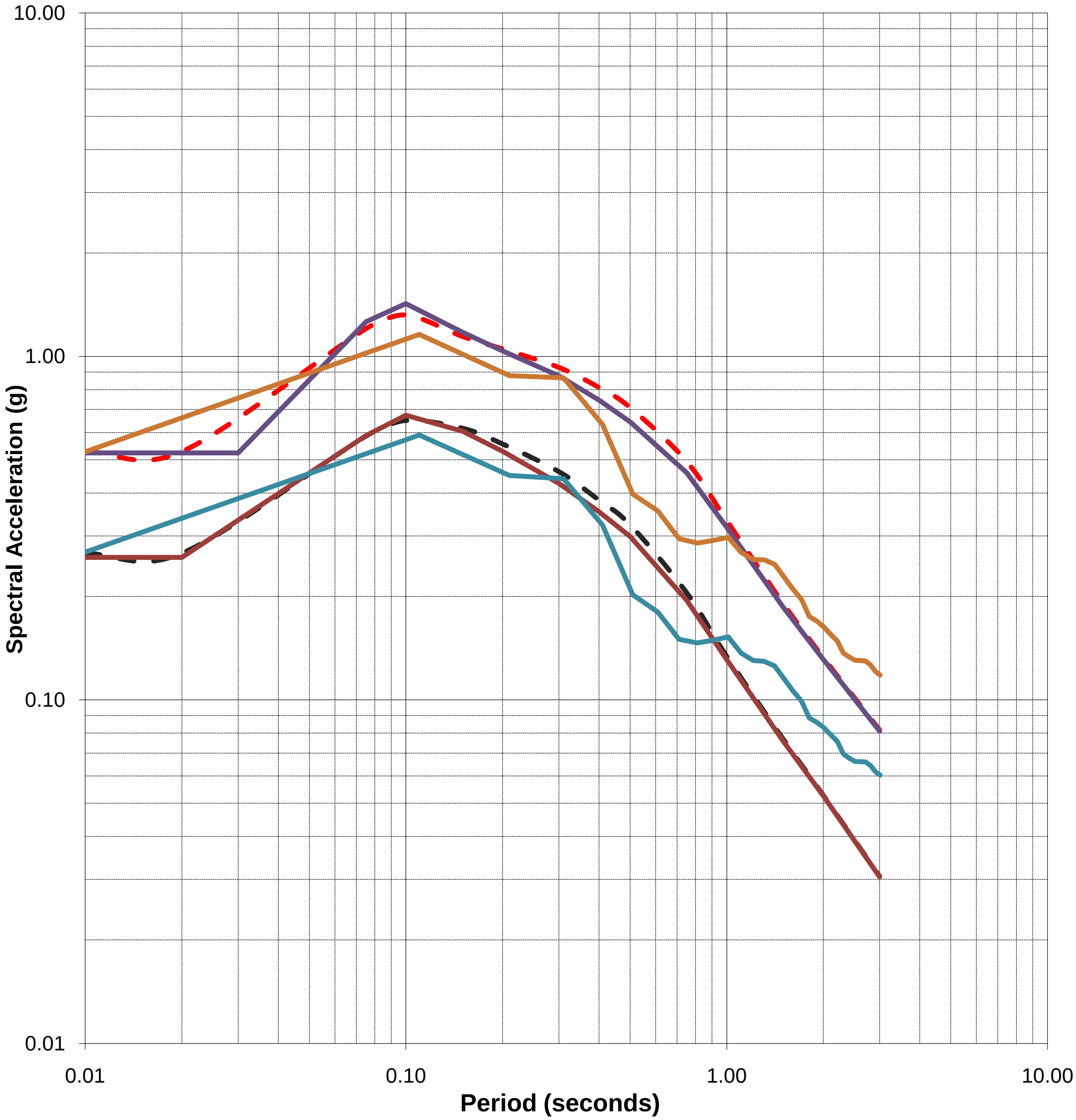
9851-006-00 10/28/2009 KHC:Site Specific Response Spectra.xls

GeoEngineers
Earth Science + Technology

5% Damped Horizontal Response Spectra

Figure 13

VA Hospital, Seattle
Average Vertical Response Spectrum of 7 Scaled Motions
(Nursing Tower)



- Site Specific BSE-1 Ground Surface Vertical Response Spectrum
- Site Specific BSE-2 Ground Surface Vertical Response Spectrum
- Site Specific BSE-1 Vertical Response Spectrum at 15 feet Below Ground Surface
- Site Specific BSE-2 Vertical Response Spectrum at 15 feet Below Ground Surface
- Average Vertical Response Spectrum of 7 Scaled Motions for BSE-1
- Average Vertical Response Spectrum of 7 Scaled Motions for BSE-2

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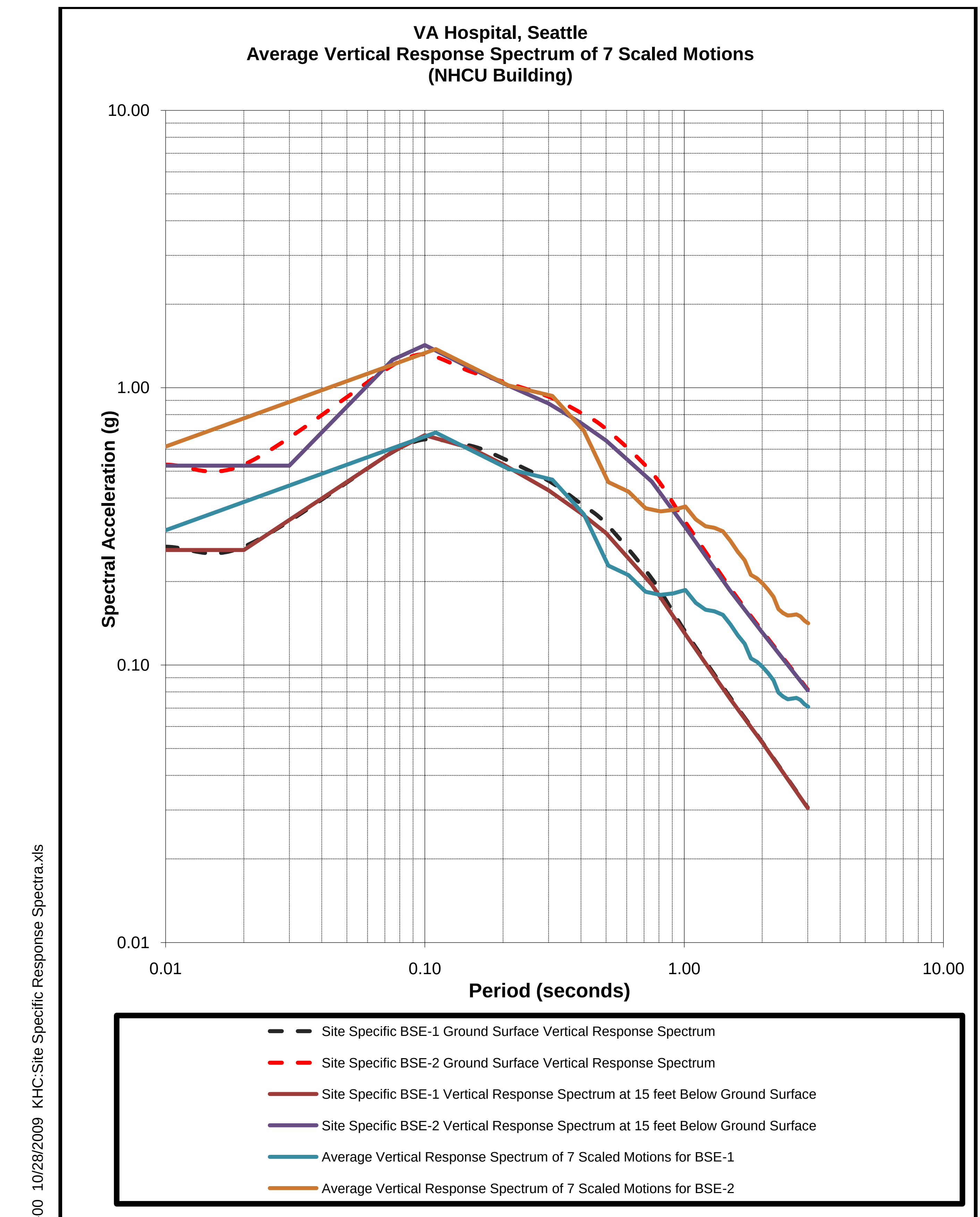
5% Damped Vertical Response Spectra

Figure 14

9851-006-00 10/28/2009 KHC:Site Specific Response Spectra.xls



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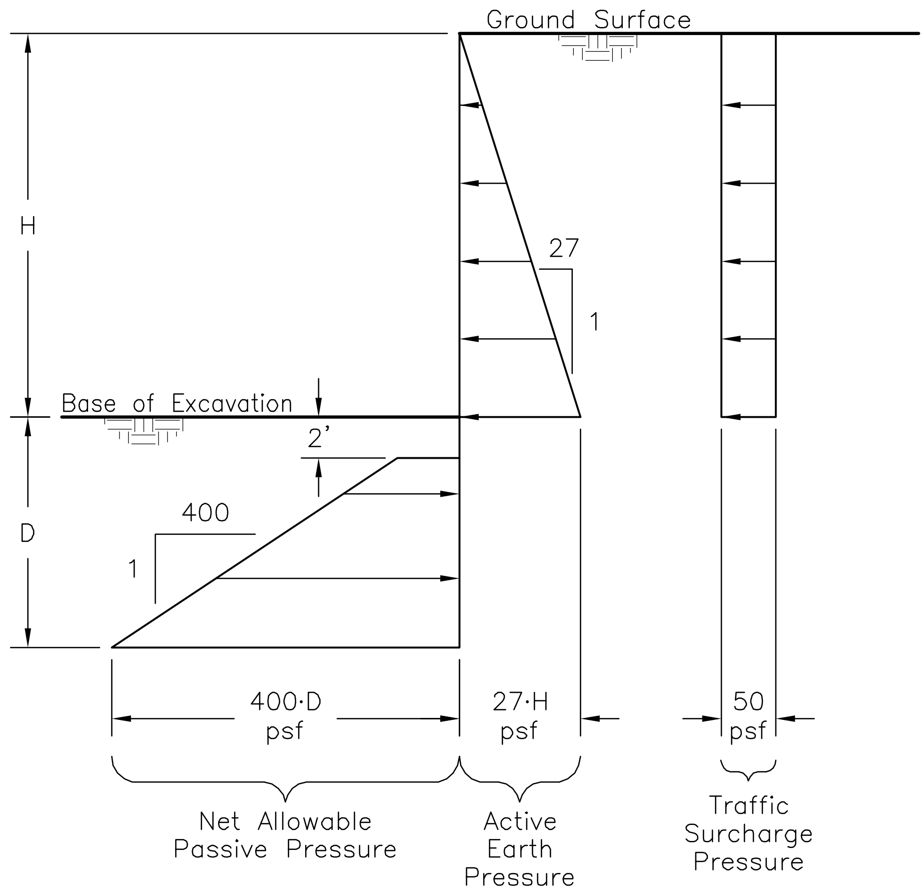
5% Damped Vertical Response Spectra

Figure 15



NOT TO SCALE

Legend

H = Height of Excavation, Feet

D = Soldier Pile Embedment, Feet

Notes:

1. Active earth pressure and surcharge pressure act over the pile spacing above the base of the excavation.
2. Passive earth pressure acts over 2.5 times the concreted diameter of the soldier pile, or the pile spacing, whichever is less.
3. Passive pressure includes a factor of safety of 1.5.
4. This pressure diagram is appropriate for temporary soldier pile walls. If additional surcharge loading (such as from soil stockpiles, excavators, dumptrucks, cranes, or concrete trucks) is anticipated, GeoEngineers should be consulted to provide revised surcharge pressures.

Earth Pressure Diagram
Temporary Cantilever Soldier Pile Wall

New Mental Health Building/Seismic Retrofit of Nursing Tower/New Parking Garage
Seattle, Washington

GeoEngineers

Figure 16

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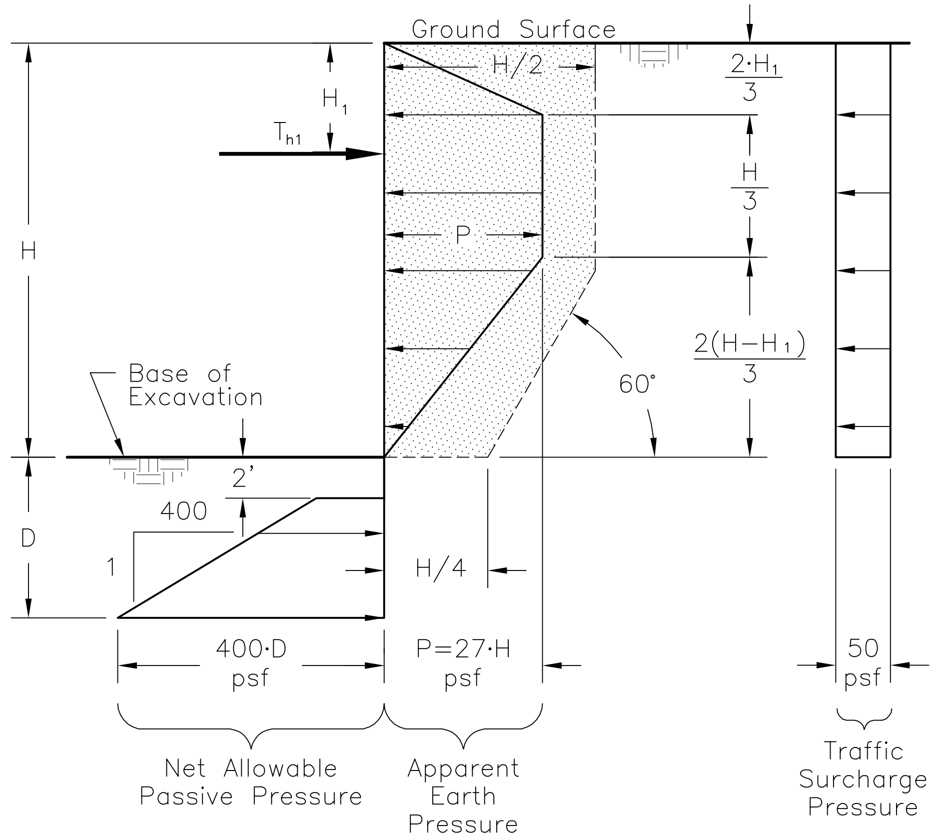
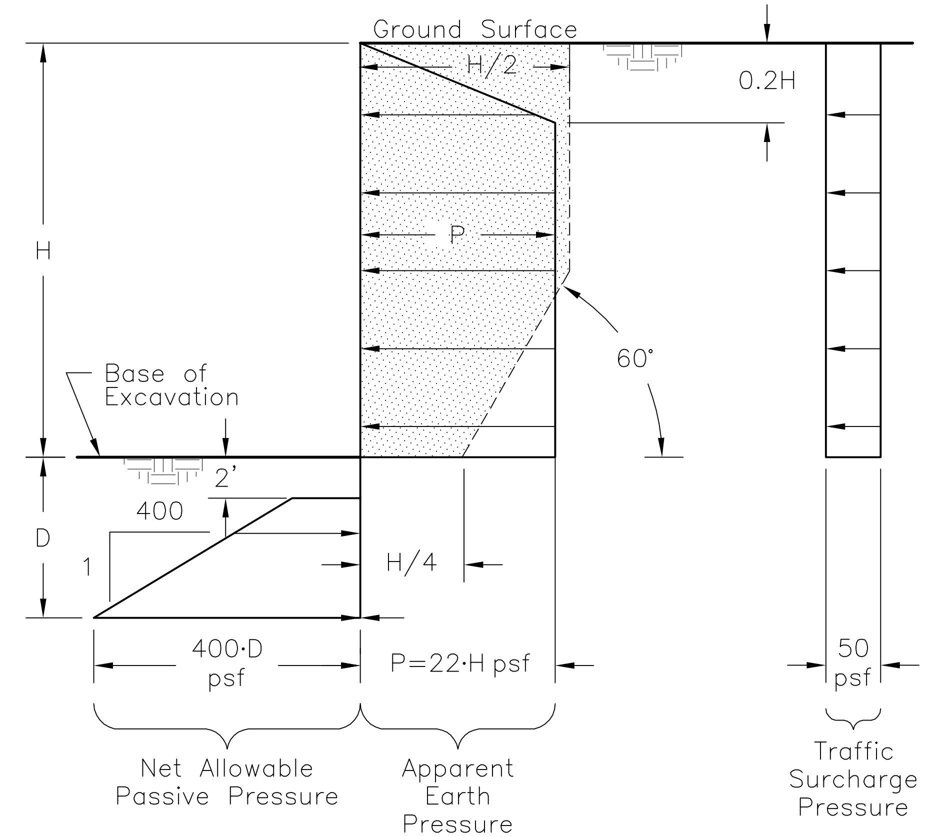
SOLDIER PILE WALL WITH ONE LEVEL OF TIEBACKS



SOLDIER PILE WALL WITH MULTIPLE LEVELS OF TIEBACKS



NOT TO SCALE

Notes:
1. Apparent earth pressure and surcharge act over the pile spacing above the base of the excavation.
2. Passive earth pressure acts over 2.5 times the concreted diameter of the soldier pile, or the pile spacing, whichever is less.
3. Passive pressure includes a factor of safety of 1.5
4. Additional surcharge from footings of adjacent buildings should be included in accordance with recommendations provided on Figure 4.
5. This pressure diagram is appropriate for temporary soldier pile and tieback walls. If additional surcharge loading (such as from soil stockpiles, excavators, dumptrucks, cranes, or concrete trucks) is anticipated, GeoEngineers should be consulted to provide revised surcharge pressures.

Legend

No Load Zone

H = Height of Excavation, Feet

D = Soldier Pile Embedment Depth, Feet

H_1 = Distance From Ground Surface to Uppermost Tieback, Feet

T_{h1} = Horizontal Load in Uppermost Ground Anchor

P = Maximum Apparent Earth Pressure Pounds per Square Foot

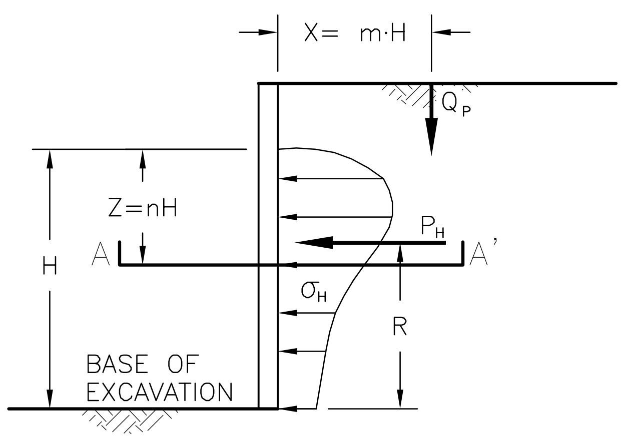
Earth Pressure Diagrams
Temporary Soldier Pile & Tieback Wall

New Mental Health Building/Seismic Retrofit of Nursing Tower/New Parking Garage
Seattle, Washington

GEOENGINEERS

Figure 17

LATERAL EARTH PRESSURE FROM POINT LOAD, Q_P (SPREAD FOOTING)



FOR m ≤ 0.4

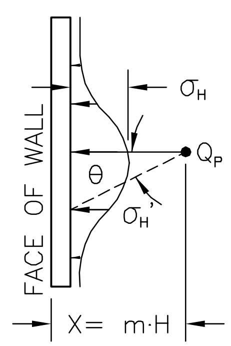
$$\sigma_H = \frac{0.28 Q_P n^2}{H^2(0.16+n^2)^3}$$

FOR m > 0.4

$$\sigma_H = \frac{1.77 Q_P m^2 n^2}{H^2(m^2+n^2)^3}$$

$$\sigma_H' = \sigma_H \cos^2(1.1\theta)$$

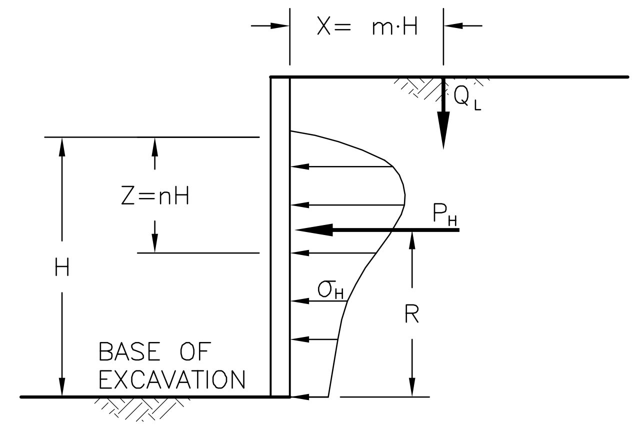
M	$P_H\left(\frac{H}{Q_P}\right)$	R
0.2	0.78	0.59H
0.4	0.78	0.59H
0.6	0.45	0.48H



SECTION A–A'

Pressures from Point Load Q_P

LATERAL EARTH PRESSURE FROM LINE LOAD, Q_L (CONTINUOUS WALL FOOTING)



FOR m ≤ 0.4

$$\sigma_H = \frac{0.2 n \cdot Q_L}{H(0.16+n^2)^2}$$

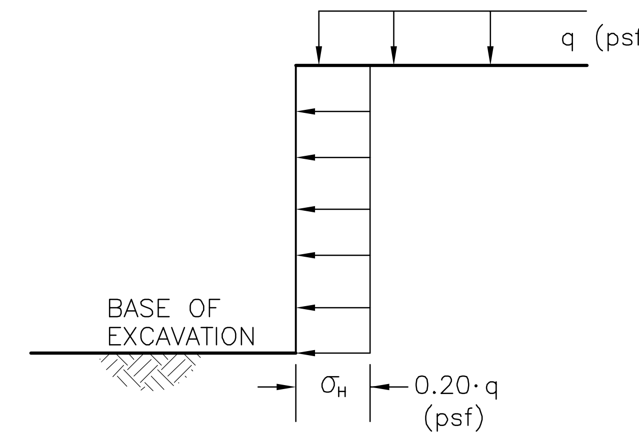
FOR m > 0.4

$$\sigma_H = \frac{1.28 m^2 n \cdot Q_L}{H(m^2+n^2)^2}$$

$$\text{RESULTANT } P_H = \frac{0.64 Q_L}{(m^2+1)}$$

M	R
0.1	0.60H
0.3	0.60H
0.5	0.56H
0.7	0.48H

UNIFORM SURCHARGES, q (FLOOR LOADS, LARGE FOUNDATION ELEMENTS OR TRAFFIC LOADS)



σ_H = LATERAL SURCHARGE PRESSURE FROM UNIFORM SURCHARGE

Definitions:

- Q_P = Point load in pounds
- Q_L = Line load in pounds/foot
- H = Excavation height below footing, feet
- σ_H = Lateral earth pressure from surcharge, psf
- q = Surcharge pressure in psf
- θ = Radians
- σ_H' = Distribution of σ_H in plan view
- P_H = Resultant lateral force acting on wall, pounds
- R = Distance from base of excavation to resultant lateral force, feet

Notes:
1. Procedures for estimating surcharge pressures shown above are based on Manual 7.02 Naval Facilities Engineering Command, September 1986 (NAVFAC DM 7.02).
2. Lateral earth pressures from surcharge should be added to earth pressures presented on Figure 3.
3. See report text for where surcharge pressures are appropriate.

Recommended Surcharge Pressure

New Mental Health Building/Seismic Retrofit of Nursing Tower/New Parking Garage
Seattle, Washington

GEOENGINEERS

Figure 18

APPENDIX A
Field Explorations

APPENDIX A
FIELD EXPLORATIONS

General

Subsurface conditions were explored at the site by drilling sixteen borings (GEI-1 through GEI-16). The borings were completed to depths between 11½ and 62 feet below the existing ground surface. The drilling was performed by Geologic Drill on September 24 and 25, 2009 (GEI-1 through GEI-8) and September 1 and 2, 2010 (GEI-9 through GEI-16).

The locations of the explorations were estimated in the field by measuring distances from site features through taping/pacing in the field. The approximate exploration locations are shown on the Site Plan, Figure 2. Boring elevations were estimated based on a topographic map prepared by PLS, Inc. dated October 15, 2009.

Borings

Borings were completed using trailer-mounted, continuous-flight, hollow-stem auger drilling equipment. The borings were continuously monitored by a geotechnical engineer from our firm who examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions and prepared a detailed log of each exploration.

The soils encountered in the borings were generally sampled at 2½- or 5-foot vertical intervals with a 2-inch outside diameter split-barrel standard penetration test (SPT) sampler. The samples were obtained by driving the sampler 18 inches into the soil with a 140-pound hammer with a rope and cathead free-falling 30 inches. The number of blows required for each 6 inches of penetration was recorded. The blow count ("N-value") of the soil was calculated as the number of blows required for the final 12 inches of penetration. This resistance, or N-value, provides a measure of the relative density of granular soils and the relative consistency of cohesive soils. Where very dense soil conditions preclude driving the full 18 inches, the penetration resistance for the partial penetration was entered on the logs. The blow counts are shown on the boring logs at the respective sample depths.

Soils encountered in the borings were visually classified in general accordance with the classification system described in Figure A-1. A key to the boring log symbols is also presented in Figure A-1. The logs of the borings are presented in Figures A-2 through A-17. The boring logs are based on our interpretation of the field and laboratory data and indicate the various types of soils and groundwater conditions encountered. The logs also indicate the depths at which these soils or their characteristics change, although the change may actually be gradual. If the change occurred between samples, it was interpreted. The densities noted on the boring logs are based on the blow count data obtained in the borings and judgment based on the conditions encountered.

Observations of groundwater conditions were made during drilling. The groundwater conditions encountered during drilling are presented on the boring logs. Groundwater conditions observed during drilling represent a short-term condition and may or may not be representative of the long-term groundwater conditions at the site. Groundwater conditions observed during drilling should be considered approximate.

Geophysical Test Casing Installation

A representative of GeoEngineers observed the installation of a blank casing for downhole geophysical testing in boring GEI-7. The casing was constructed using 2-inch-diameter polyvinyl chloride (PVC) casing. The casing was installed to a depth of 61.5 feet. The casing was backfilled using a mixture of portland cement, bentonite and water. The casing was protected by installing a flush-mount steel monument set in concrete.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	CC	Cement Concrete
	AC	Asphalt Concrete
	CR	Crushed Rock/ Quarry Spalls
	TS	Topsoil/ Forest Duff/Sod

Measured groundwater level in exploration, well, or piezometer

Groundwater observed at time of exploration

Perched water observed at time of exploration

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Material Description Contact

Distinct contact between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

A "P" indicates sampler pushed using the weight of the drill rig.

Laboratory / Field Tests

%F	Percent fines
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
OC	Organic content
PM	Permeability or hydraulic conductivity
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen
NT	Not Tested

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

KEY TO EXPLORATION LOGS

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FIGURE A-1

Drilled	Start 9/25/2009	End 9/25/2009	Total Depth (ft) 30	Logged By BPD Checked By DPC	Driller Geologic Drill	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) Vertical Datum: 328.0	Hammer Data: Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment: XL-Trailer
Easting (X): 1276551.336 Northing (Y): 208706.198	System Datum: NAD83	Groundwater — Date Measured: 9/25/2009; Depth to Water (ft): 20; Elevation (ft): 308.0
Notes: HSA 7 1/4" OD; 3 1/4" ID		

Elevation (feet)	Depth (feet)	FIELD DATA: Recovered (in)	Blows/foot	Sample Name Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0					CC	4 inch Portland Cement concrete pavement			
						SM	Light brown silty fine sand (medium dense, moist) (fill)			
325			11	1						
						ML	Light brown silt with sand (stiff, moist to wet)			
	5	18	12	2				29		%F=83
						SP-SM	Light brown fine sand with silt (medium dense to dense, moist) (glacially consolidated soils)			
320		18	14	3				9		%F=10
	10	18	39	4						
315						SM	Light brown silty fine sand (dense, moist)			
	15	18	46	5						
310										
	20	18	43	6	▽	SP-SM	Light brown fine to medium sand with silt (dense, wet)	24		Rods wet from 20 feet %F=5
305						CH	Gray clay with interbedded sand layers (very stiff, moist to wet)			Harder drilling
	25	18	30	7						Very slow drilling from 25 feet
300						CLAY-STONE	Dark gray claystone (hard, moist) (bedrock)			
		18	33	8				35		AL
	30									

Note: See Figure A-1 for explanation of symbols.

Redmond: Date:10/30/09 Path:W:\REDMOND\PROJECTS\9\9851006\GINT\9851006.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_STANDARD

Log of Boring GEI-1

GEOENGINEERS

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Figure A-2
Sheet 1 of 1

Drilled	Start 9/24/2009	End 9/24/2009	Total Depth (ft)	23	Logged By BPD / Checked By DPC	Driller: Geologic Drill	Drilling Method: Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum	331.0	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop
Drilling Equipment	XL-Trailer		
Easting (X) / Northing (Y)	1276444.506 / 208788.108	System Datum	NAD83

Groundwater: Date Measured | Depth to Water (ft) | Elevation (ft)

Notes: HSA 7 1/4" OD; 3 1/4" ID

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
331	0				TS	8 inches topsoil			
330					SM	Gray brown silty fine to medium sand with gravel (medium dense to dense, moist) (fill)			
		18	40	1					
325	5	18	21	2					
		18	73	3	SM	Gray silty fine to medium sand with gravel and occasional cobbles (very dense, moist) (glacial till) (glacially consolidated soils)			(till-like fabric)
320	10	12	50/6"	4			8		%F=23
		1	50/1"	5					
315	15	6	50/6"	6					
		6	50/6"	7					
310	20								
		6	50/6"	8					

Groundwater was not encountered

Note: See Figure A-1 for explanation of symbols.

Log of Boring GEI-2

GeoEngineers

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Figure A-3
Sheet 1 of 1

Drilled Start 9/24/2009 End 9/24/2009	Total Depth (ft) 23.5	Logged By BPD Checked By DPC	Driller Geologic Drill	Drilling Method Hollow-Stem Auger	
Surface Elevation (ft) Vertical Datum 325.5		Hammer Data Rope & Cathead 140 (lbs) / 30 (in) Drop		Drilling Equipment XL-Trailer	
Easting (X) 1276219.353 Northing (Y) 208540.581		System Datum NAD83		Groundwater	
Notes: HSA 7 1/4" OD; 3 1/4" ID				Date Measured: 9/24/2009 Depth to Water (ft): 15	Elevation (ft): 310.5

FIELD DATA

Elevation (feet)	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample / Sample Name Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
325	0					TS	3 inches topsoil			
						SM	Gray/brown silty fine to medium sand (medium dense, moist) (fill)			
		18	27	1						
320	5					SM	Gray/brown silty fine sand (dense, moist) (glacially consolidated soils)			
		18	37	2				16		%F=25
315	10					ML	Gray silt with fine sand and occasional gravel (hard, moist)			Rough drilling
		12	50/6"	3						
310	15				▽	SM	Gray silty fine to medium sand with occasional gravel (very dense, wet)			
		12	50/4"	5						
305	20					SM	Gray silty fine to medium sand with gravel (very dense, moist)			Till-like
		11	50/5"	6						

Note: See Figure A-1 for explanation of symbols.

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Log of Boring GEI-3

GEOENGINEERS

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Figure A-4
Sheet 1 of 1

Drilled	Start 9/25/2009	End 9/25/2009	Total Depth (ft) 42.5	Logged By BPD, Checked By DPC	Driller Geologic Drill	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum: 315.0	Hammer Data: Rope & Cathead, 140 (lbs) / 30 (in) Drop	Drilling Equipment: XL-Trailer	
Easting (X): 1275651.509 / Northing (Y): 208814.264	System Datum: NAD83	Groundwater	
Notes: HSA 7 1/4" OD; 3 1/4" ID		Date Measured: 9/25/2009	Depth to Water (ft): 29; Elevation (ft): 286.0

Elevation (feet)	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0							AC	3 inches asphalt concrete pavement			No base course
								SM	Light brown silty fine sand (medium dense, moist) (fill)			Easy drilling
		18	18		1							
310	5											
		18	69		2			SM	Gray/brown silty fine sand with occasional gravel (medium dense to very dense, moist to wet) (glacially consolidated soils)			Harder drilling at 7 feet
305	10											Very hard drilling at 11 feet
		18	29		3				Grades to brown silty fine sand			
300	15								Grades to gray			
		18	42		4					24		%F=43
295	20											
		18	63		5							
290	25							CH/MH	Gray silt/clay (very stiff to hard, moist)			
		18	33		6	▽						Horizontal bedding; massive fabric
285	30											
		18	30		7							
280	35											

Note: See Figure A-1 for explanation of symbols.

Log of Boring GEI-4

GEOENGINEERS

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	35												
			18	52		8	▽		Siltstone	Gray siltstone (hard, wet) (bedrock)			Harder drilling at 37 feet Perched groundwater encountered
													Highly fractured
275	40												
			12	50/6"		9							Very slow drilling at 41 feet

Note: See Figure A-1 for explanation of symbols.

Log of Boring GEI-4 (continued)

GEOENGINEERS

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Drilled	Start: 9/25/2009	End: 9/25/2009	Total Depth (ft)	29	Logged By: BPD / Checked By: DPC	Driller: Geologic Drill	Drilling Method: Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum	319.5	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	XL-Trailer
Easting (X) / Northing (Y)	1275781.564 / 209279.189	System Datum	NAD83	Groundwater	Date Measured / Depth to Water (ft) / Elevation (ft)

Notes: HSA 7 1/4" OD; 3 1/4" ID

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0								AC	2 1/2 inches asphalt concrete pavement			
									SM	Light brown silty fine sand with occasional gravel (very dense, moist) (glacially consolidated soils)			No base course
			18	60		1							Oxidation staining
315	5												Rough drilling at 4 feet
									SP-SM	Light brown fine sand with silt (dense to very dense, moist)			
			18	42		2					9		%F=10
310	10												
			18	45		3					9		%F=5
305	15												
			18	56		4							
300	20								ML/CL	Gray silt/clay (hard, moist)			
			18	64		5							
295	25												
			18	33		6					34		Horizontal bedding AL

Groundwater was not encountered

Note: See Figure A-1 for explanation of symbols.

Log of Boring GEI-5

GeoEngineers

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Drilled	Start 9/25/2009	End 9/25/2009	Total Depth (ft)	25	Logged By BPD / Checked By DPC	Driller Geologic Drill	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum: 338.5	Hammer Data: Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment: XL-Trailer	
Easting (X): 1276033.255 / Northing (Y): 209438.329	System Datum: NAD83	Groundwater — Date Measured / Depth to Water (ft) / Elevation (ft)	
Notes: HSA 7 1/4" OD; 3 1/4" ID		No groundwater was encountered	

FIELD DATA

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0								AC	1 1/2 inches asphalt concrete pavement			1" gravel base course below asphalt concrete pavement
									SM	Gray/brown silty fine to medium sand with gravel (dense to very dense, moist) (glacially consolidated soils)			
335			18	83		1							
	5												Hard drilling 5 to 10 feet
330			1	50/1"		2							
	10												Easy drilling at 10 feet
325			18	48		3							
	15		18	50		4			SP-SM	Light brown fine sand with silt (very dense, moist)			Interbedded silt layers
320			18	68		5					6		%F=9
	20												
315			18	63		6							
	25												

Groundwater was not encountered

Note: See Figure A-1 for explanation of symbols.

Log of Boring GEI-6

GEOENGINEERS

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Figure A-7
Sheet 1 of 1

Drilled	Start: 9/24/2009	End: 9/24/2009	Total Depth (ft)	62	Logged By: BPD / Checked By: DPC	Driller	Geologic Drill	Drilling Method	Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum	349.0	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	XL-Trailer
Easting (X) / Northing (Y)	1276203.888 / 209191.737	System Datum	NAD83	Groundwater	Date Measured / Depth to Water (ft) / Elevation (ft)

Notes: HSA 7 1/4" OD; 3 1/4" ID

Elevation (feet)	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0							CC	2 inches recycled concrete surfacing			
								SM	Gray/brown silty fine sand with gravel (medium dense to very dense, moist) (fill)			
		4	50/6"		1							Rock in shoe
345	5	18	37		2							
		18	32		3							
340	10	18	14		4							
335								SP-SM	Light brown fine sand with silt (dense, moist) (glacially consolidated soils)			
	15	18	40		5							
330	20	18	24		6			MH	Gray/brown silt with sand (very stiff, moist to wet)			
325								SM	Light brown fine sand with silt (dense, moist)			
	25	18	39		7							
320								MH	Gray/brown silt with sand (hard, moist)			Oxidation staining
	30	18	41		8							
								SM	Gray/brown silty fine sand (dense, moist)			
315								ML	Gray silt (hard, moist)			
	35											

Note: See Figure A-1 for explanation of symbols.

Log of Boring GEI-7

GeoEngineers

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Figure A-8
Sheet 1 of 2

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name	Group Classification	Material Description	Moisture Content, %	Dry Density, (pcf)	Remarks
	35	18	68	9					
310	40	12	50/6"	10	SM/ML	Gray silty fine sand with gravel and gray silt with sand (hard, moist)			
305					SM	Gray silty fine sand with occasional gravel (very dense, moist)			Till-like
	45	18	50/6"	11					
300	50	6	50/6"	12	Siltstone	Gray siltstone/claystone (hard, moist to wet) (bedrock)			
295	55	12	50/6"	13					15 minutes for 5 feet
290	60	5.5	50/5.5"	14					
		0	50/0.5"	15					

Practical refusal at 62'
Installed 2" PVC casing and backfilled with 50/50 bentonite/cement mixture for shear wave velocity testing
Groundwater was not encountered

Note: See Figure A-1 for explanation of symbols.

Log of Boring GEI-7 (continued)

GeoEngineers

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Drilled	Start 9/25/2009	End 9/25/2009	Total Depth (ft)	34	Logged By BPD / Checked By DPC	Driller	Geologic Drill	Drilling Method	Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum: 332.5	Hammer Data: Rope & Cathead, 140 (lbs) / 30 (in) Drop	Drilling Equipment: XL-Trailer	
Easting (X): 1275917.858 / Northing (Y): 208989.71	System Datum: NAD83	Groundwater	
Notes: HSA 7 1/4" OD; 3 1/4" ID		Date Measured: 9/25/2009	Depth to Water (ft): 29 / Elevation (ft): 303.5

FIELD DATA

Elevation (feet)	Depth (feet)	Interval / Recovered (in)	Blows/foot	Collected Sample / Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0					AC	3 inches asphalt concrete pavement			
						SM	Gray/brown silty fine to medium sand with occasional gravel (loose, moist to wet) (fill)			
330		18	5	1						
	5	18	11	2		ML	Gray/brown sandy silt with occasional gravel and organic matter (stiff, moist to wet)			
325		18	6	3		SM	Gray and brown silty fine to medium sand with gravel (very loose to loose, moist to wet)			
	10	18	4	4						
320		18	4	5						
	15	18	6	6						
315		18	29	7		SM	Light brown silty fine to medium sand and occasional gravel (medium dense to very dense, moist) (glacially consolidated soils)	11		Driller noted change in drilling at 17 feet %F=30
	20						Grades to gray/brown silty fine to medium sand with gravel			
310			50/6"	8						
	25					ML	Gray and brown sandy silt (hard, moist to wet)			
305		18	43	9	▽					
	30					SM	Light brown silty fine sand (very dense, wet)			Rods wet at 30 feet
300		18	66	10						

Note: See Figure A-1 for explanation of symbols.

Log of Boring GEI-8

GEOENGINEERS

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Drilled	Start 9/1/2010	End 9/1/2010	Total Depth (ft)	31.5	Logged By NCS Checked By DPC	Driller	Geologic Drill	Drilling Method	Hollow-Stem Auger

Surface Elevation (ft) Vertical Datum	322.0	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop
Drilling Equipment	XL-Trailer		
Easting (X) Northing (Y)	1275852.968 209471.565	System Datum	NAD83

Notes: Auger Data: 3¼ inches I.D; 8 inches O.D

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
9/1/2010	18.0	304.0

FIELD DATA

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0				AC	3 inches asphalt concrete pavement			
320					GP-GM	Brown silty fine to medium sand with gravel and occasional cobbles (medium dense, moist) (fill)			
		16	33	1	SM	Grayish brown silty fine to medium sand with gravel and occasional cobbles (dense, moist) (fill)			
					ML	Grayish brown sandy silt with occasional gravel (very stiff, moist) (glacially consolidated)			
	5	18	21	2			16		%F=57
315		18	22	3					
					SM	Brown gray silty fine to medium sand with occasional gravel (medium dense, moist)			Hard drilling
310	10	18	29	4					
					SM	Grayish brown silty fine to medium sand with interbedded silt seams (½ to 1-inch thick) (medium dense, moist to wet)			
305	15	18	24	5			17		%F=21
					SP-SM	Grayish brown fine to medium sand with silt (dense, wet)			Rods wet
300	20	18	42	6			22		%F=8
					ML	Gray silt (hard, moist)			
295	25	18	50/5"	7					Massive fabric
	30	18	52	8					

Note: See Figure A-1 for explanation of symbols.

Log of Boring GEI-9

GeoEngineers

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Figure A-10
Sheet 1 of 1

Drilled	Start 9/1/2010	End 9/1/2010	Total Depth (ft)	11.5	Logged By NCS / Checked By DPC	Driller Geologic Drill	Drilling Method	Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum	327.0	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	XL-Trailer
Easting (X) / Northing (Y)	1275939.229 / 209395.807	System Datum	NAD83	Groundwater	Date Measured / Depth to Water (ft) / Elevation (ft)

Notes: Auger Data: 3¼ inches I.D; 8 inches O.D

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0								AC	2½ inches asphalt concrete pavement			
									GP-GM	Brown fine gravel base course with silt and sand (medium dense, moist)			Hard drilling
325									SM	Brown silty fine to medium sand with gravel (medium dense, moist) (fill)			
			18	56		1			SM	Gray brown silty fine to medium sand with gravel and occasional cobbles (very dense, moist) (glacially consolidated soils)			
	5		17	50/5"		2							
320									ML	Gray brown interbedded silty fine to medium sand with occasional gravel (dense to very stiff, moist)			
	10		18	36		3					19		%F=67

Note: See Figure A-1 for explanation of symbols.

Log of Boring GEI-10

GeoEngineers

Project: Mental Health Building/Garage/Nursing Tower

Project Location: Seattle, Washington

Project Number: 9851-006-00

Figure A-11

Sheet 1 of 1

Drilled	Start 9/1/2010	End 9/1/2010	Total Depth (ft)	31.5	Logged By NCS Checked By DPC	Driller	Geologic Drill	Drilling Method	Hollow-Stem Auger

Surface Elevation (ft) Vertical Datum	331.0	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	XL-Trailer
Easting (X) Northing (Y)	1275965.244 209292.829	System Datum	NAD83	Groundwater	

Notes: Auger Data: 3¼ inches I.D; 8 inches O.D

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
9/1/2010	25.0	306.0

FIELD DATA

Elevation (feet)	Depth (feet)	Interval Recovered (in)	Blows/foot	Sample Name Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
331	0					AC	2 inches asphalt concrete pavement			
						GP-GM	Brown fine to coarse gravel base course with silt and sand (medium dense, moist)			
330						SM	Brown and gray silty fine to coarse sand with gravel and occasional cobbles (dense, moist) (fill)			
		16	36	1						
325	5	18	19	2		SM	Brown silty fine to medium sand with gravel and occasional cobbles (medium dense, moist) (weathered glacially consolidated deposit)			
						SM	Brown silty fine to medium sand with gravel and occasional cobbles (very dense, moist) (glacially consolidated soils)			Harder drilling
		18	51	3						
320	10	18	50/5"	4						
315	15	18	50/3"	5						
						SP-SM	Brown fine to medium sand with silt (very dense, moist)			
310	20	18	47	6a 6b						
						ML	Brown silt with occasional sand (hard, moist to wet)			
305	25	18	32	7	∇	SP	Gray fine to medium sand with trace silt (dense, wet)	22		%F=4
						ML	Gray silt (hard, moist)			
300	30	18	58	8						Massive fabric

Note: See Figure A-1 for explanation of symbols.

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Log of Boring GEI-11

GEOENGINEERS

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Figure A-12
Sheet 1 of 1

Drilled	Start 9/1/2010	End 9/1/2010	Total Depth (ft) 26.5	Logged By NCS / Checked By DPC	Driller Geologic Drill	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum: 324.5	Hammer Data: Rope & Cathead, 140 (lbs) / 30 (in) Drop	Drilling Equipment: XL-Trailer	
Easting (X): 1275854.038 / Northing (Y): 209177.551	System Datum: NAD83	Groundwater	
		Date Measured / Depth to Water (ft) / Elevation (ft)	

Notes: Auger Data: 3¼ inches I.D; 8 inches O.D

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0								AC	2½ inches asphalt concrete pavement			Rough drilling
									SM	Brown silty fine to medium sand with gravel (medium dense, moist) (fill)			
									SM	Gray brown silty fine to medium sand with gravel and occasional cobbles (very dense, moist) (glacially consolidated soils)			
			18	50/6"		1							
320	5		12	50/6"		2							
			9	50/3"		3							
315	10		5	50/5"		4							Gravel in sampler
310	15		12	58		5					10		%F=17 Easier drilling
305									SP-SM	Brown fine to medium sand with silt (dense, moist)			
	20		18	48		6							
										Moisture in sample increases			
300	25		18	48		7					22		%F=10 Oxidation staining

Note: See Figure A-1 for explanation of symbols.

Log of Boring GEI-12

GeoEngineers

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Figure A-13
Sheet 1 of 1

Drilled	Start 9/1/2010	End 9/1/2010	Total Depth (ft)	41.5	Logged By NCS Checked By DPC	Driller Geologic Drill	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) Vertical Datum: 336.5	Hammer Data: Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment: XL-Trailer	
Easting (X): 1275984.844 Northing (Y): 209129.49	System Datum: NAD83	Groundwater	Date Measured / Depth to Water (ft) / Elevation (ft)

Notes: Auger Data: 3¼ inches I.D; 8 inches O.D

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0				AC	3 inches asphalt concrete pavement			
335					SM	Dark brown and gray silty fine to medium sand with gravel and occasional cobbles (loose to medium dense, moist) (fill)			No gravel base course observed
		18	8	1					
	5	118	19	2					Blowcount may not be representative; SPT sampler on gravel
330		6	53	3		Grades to grayish brown			
	10	15	27	4	SM	Brown to gray silty fine to medium sand with gravel and occasional cobbles (medium dense to very dense, moist) (glacially consolidated soils)			
325									
	15	1	50/6"	5					
320					SM	Light brown silty fine to medium sand (dense, moist)			
	20	15	48	6			8		%F=27
315					SP-SM	Light brown fine to medium sand with silt and occasional gravel (very dense, moist)			
	25	18	69	7			5		%F=6
310									
	30	18	63	8					
305							16		%F=9
	35				ML	Gray silt (hard, moist)			

Note: See Figure A-1 for explanation of symbols.

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Log of Boring GEI-13

GEOENGINEERS

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Figure A-14
Sheet 1 of 2

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
300	35		18	54		9							Massive fabric
295	40		18	38		10							

Note: See Figure A-1 for explanation of symbols.

Log of Boring GEI-13 (continued)

GEOENGINEERS

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Drilled	Start 9/1/2010	End 9/1/2010	Total Depth (ft) 26.5	Logged By NCS / Checked By DPC	Driller Geologic Drill	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum	318.0	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	XL-Trailer
Easting (X) / Northing (Y)	1275727.284 / 208987.752	System Datum	NAD83	Groundwater	

Notes: Auger Data: 3¼ inches I.D; 8 inches O.D

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
9/1/2010	20.0	298.0

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
318	0					AC	1½ inches asphalt concrete pavement			1-inch of gravel base course below ACP
						SM	Dark gray to brown silty fine to medium sand with gravel and occasional cobbles (loose to medium dense, moist) (fill)			
315		18	16	1						
	5	18	9	2						
310		12	21	3			Grades to blue gray and silt content increases			
	10	8	19	4						
						SM	Gray brown silty fine to medium sand with gravel and occasional cobbles (medium dense, moist) (glacially consolidated soils)			
305						SP-SM	Brown fine to medium sand with silt and occasional gravel (very dense, moist)			
	15	18	55	5						
300						SM	Gray silty fine to medium sand with occasional gravel (very dense, wet)			
	20	18	52	6	▽			19		%F=49
295						SM	Gray silty fine to medium sand (very dense, wet)			
	25	18	65	7						
						ML	Gray silt with trace fine sand (hard, moist)			Massive fabric

Note: See Figure A-1 for explanation of symbols.

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Log of Boring GEI-14

GEOENGINEERS

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Figure A-15
Sheet 1 of 1

Drilled	Start 9/1/2010	End 9/1/2010	Total Depth (ft) 16.5	Logged By NCS / Checked By DPC	Driller Geologic Drill	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum	321.5	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	XL-Trailer
Easting (X) / Northing (Y)	1275841.144 / 208862.648	System Datum	NAD83	Groundwater	

Notes: Auger Data: 3¼ inches I.D; 8 inches O.D

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
9/1/2010	15.0	306.5

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
321.5	0					AC	2 inches asphalt concrete pavement			No base course observed
320						SM	Brown silty fine to medium sand with gravel and occasional cobbles (medium dense, moist) (fill)			
		18	27	1				12		SA; %F=36
	5	18	63	2						SPT sampler on gravel; Blowcount may not be representative
315						SP-SM	Light brown fine to medium sand with gravel (medium dense to dense, moist to wet) (glacially consolidated soils)			
		18	26	3				7		SA; %F=10
	10	18	46	4		SM	Light brown silty fine sand with gravel (dense, moist to wet)	16		SA; %F=36
310										
							Grades to gray			
	15	18	41	5	▽			26		SA; %F=37
305										

Note: See Figure A-1 for explanation of symbols.

Log of Boring GEI-15

GEOENGINEERS

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Drilled	Start 9/1/2010	End 9/1/2010	Total Depth (ft) 14	Logged By NCS / Checked By DPC	Driller Geologic Drill	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum	302.0	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	XL-Trailer
Easting (X) / Northing (Y)	1275680.85 / 208527.554	System Datum	NAD83	Groundwater	

Notes: Auger Data: 3¼ inches I.D; 8 inches O.D

Date Measured	Depth to Water (ft)	Elevation (ft)
9/1/2010	1.5	300.5

FIELD DATA

Elevation (feet)	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0							AC	1½ inches asphalt concrete pavement			2 inches of base course below the ACP
300						▽		ML	Dark brown and bluish gray silt with sand and occasional gravel (medium stiff, moist) (fill)			Organic matter
		18	9		1							
												Harder drilling
	5	18	10		2				Grades to sandy silt			
295								SM	Brown silty fine to medium sand with gravel and occasional cobbles (dense, moist to wet) (glacially consolidated soils)			
		4	42		3	▽						
	10	7	46		4					12		%F=25
290								SP-SM	Gray fine to medium sand with silt (very dense, moist to wet)			
		18	64		5					20		%F=38

Note: See Figure A-1 for explanation of symbols.

Log of Boring GEI-16

GEOENGINEERS

Project: Mental Health Building/Garage/Nursing Tower
Project Location: Seattle, Washington
Project Number: 9851-006-00

Appendix B
Laboratory Testing

APPENDIX B
LABORATORY TESTING

General

Soil samples obtained from the explorations were transported to GeoEngineers' laboratory and examined to confirm or modify field classifications, as well as to evaluate index properties of the soil samples. Representative samples were selected for laboratory testing consisting of the determination of the moisture content, percent fines, sieve analyses and Atterberg limits (plasticity characteristics). The tests were performed in general accordance with test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures.

The Atterberg limits test results are presented in Figure B-1. The sieve analyses test results are presented in Figure B-2. The results of the moisture content and percent passing the U.S. No. 200 sieve determinations are presented at the respective sample depths on the exploration logs in Appendix A.

Moisture Content Testing

Moisture content tests were completed in general accordance with ASTM D 2216 for representative samples obtained from the explorations. The results of these tests are presented on the exploration logs in Appendix A at the depths at which the samples were obtained.

Percent Passing U.S. No. 200 Sieve

Selected samples were "washed" through the U.S. No. 200 mesh sieve to determine the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve. These tests were conducted to verify field descriptions and to determine the fines content for analysis purposes. The tests were conducted in general accordance with ASTM D 1140, and the results are shown on the exploration logs in Appendix A at the respective sample depths.

Atterberg Limits Testing

Atterberg limits testing was performed on selected fine-grained soil samples. The tests were used to classify the soil as well as to evaluate index properties. The liquid limit and the plastic limit were estimated through a procedure performed in general accordance with ASTM D 4318. The results of the Atterberg limits testing are summarized in Figure B-1.

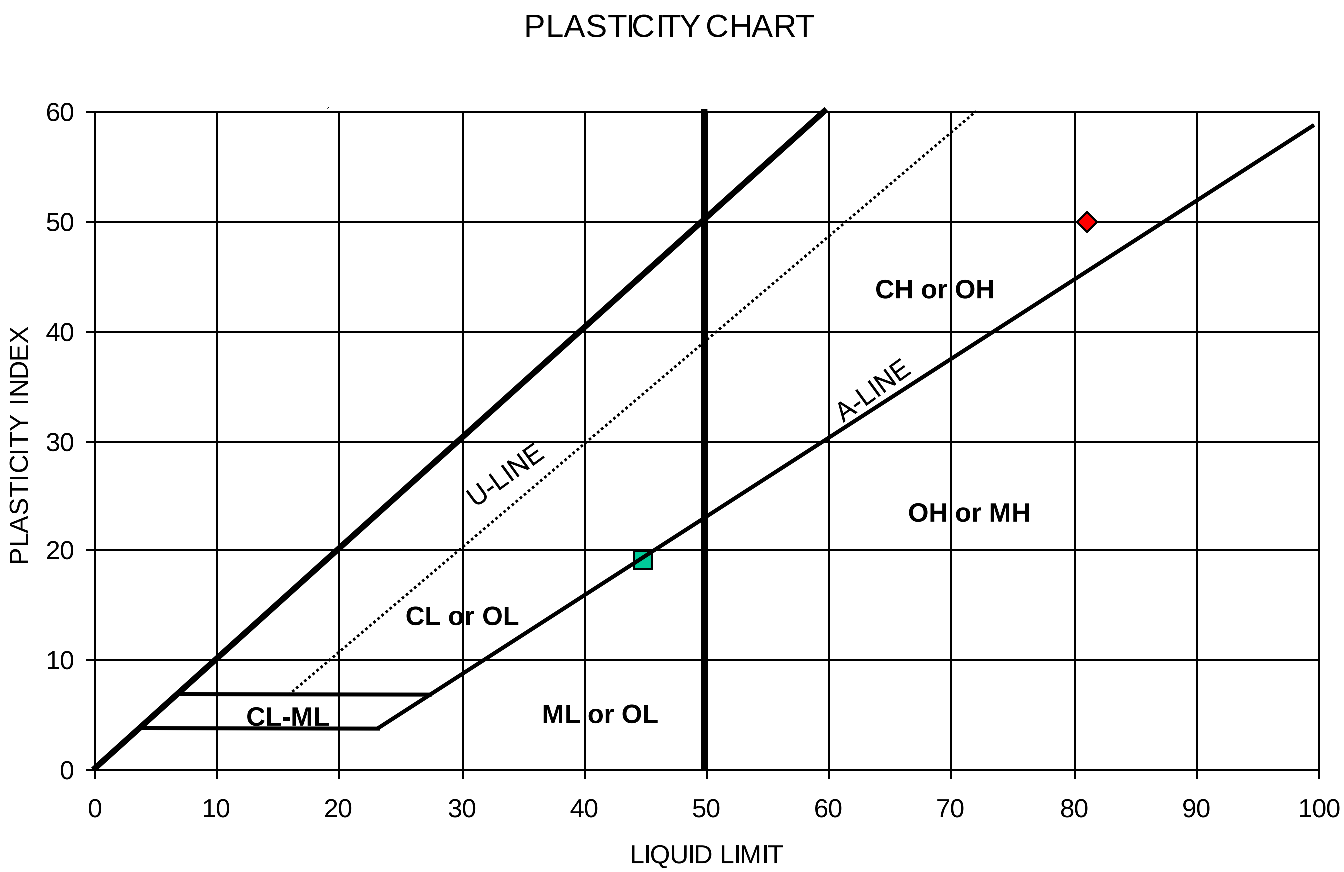
Sieve Analyses

Sieve analyses were performed on selected samples in general accordance with ASTM D 422 to determine the sample grain size distribution. The wet sieve analysis method was used to determine the percentage of soil greater than the U.S. No. 200 mesh sieve. The results of the sieve analyses were plotted and classified in general accordance with the Unified Soil Classification System (USCS) and are presented in Figure B-2.

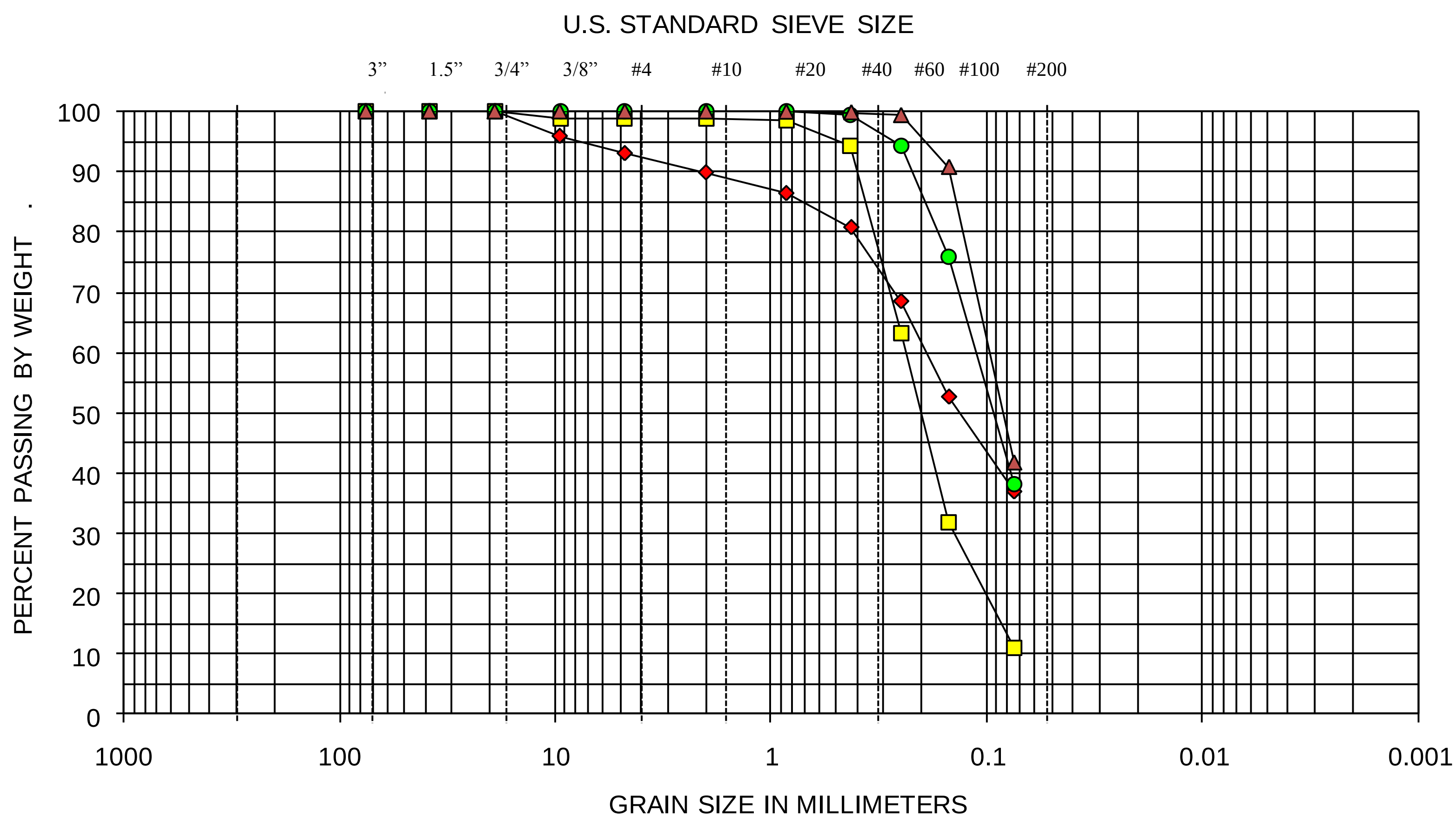
GEOENGINEERS

ATTERBERG LIMITS TEST RESULTS

FIGURE B-1



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH (FT)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	SOIL DESCRIPTION
◆	GEI-1	28 ½	35	81	50	Dark gray claystone (CH)
■	GEI-5	27 ½	34	45	19	Gray silt/clay (ML/CL)



BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	EXPLORATION NUMBER	DEPTH (ft)	SOIL CLASSIFICATION
◆	GEI-15	2.5	Brown silty sand with gravel (SM)
■	GEI-15	7.5	Light brown fine to medium sand with silt (SP-SM)
●	GEI-15	10	Light brown silty sand (SM)
▲	GEI-15	15	Gray silty sand (SM)

SIEVE ANALYSIS RESULTS

FIGURE B-2

APPENDIX C
Geophysical Measurements

GEO RECON INTERNATIONAL

applied geophysics

GeoEngineers

OCT 08 2009

Routing

File

October 1, 2009
J09-826

Geo Engineers
8410 154th Ave NE
Redmond WA 98052

Boring GEI-7
Compression and Shear Wave Velocity Measurements
Veterans Hospital, Seattle, Washington

This report presents the results of the geophysical measurements in Boring GEI-7 at the Seattle Veterans Hospital. Downhole Compression and Shear wave velocities for dynamic soil moduli determinations were measured in the borings. The field work was completed on September 29, 2009.

COMPRESSION AND SHEAR WAVE VELOCITIES

The boring was cased with 2-inch Schedule 40 PVC pipe. The 2-inch ID casing was backfilled in the borehole annulus with a bentonite/cement grout mixture.

The measured compression and shear wave velocities are presented in the tables attached to this report. The tables show the depths down the bore hole, the field measured interval times, the converted vertical downhole time arrivals, the interval vertical velocities in the boring, and the averaged velocities over the layer intervals. When the velocity boundary does not coincide with a measurement depth, the velocity calculation of that point is not accurate from the preceding point of measurement, and the velocity computation between those two points is not included in the velocity average.

Figures 1 is the time-depth plots for the boring. The plots are the corrected downhole time arrivals of the measured Compression (P) and Shear (S) wave particle motion, plotted against the depth of measurement. The velocities of the P and S waves are computed from the slopes of the time arrivals on the figures, or as the averaged velocities of the interval velocities. The figure was utilized to determine the depths of the velocity layers in the attached tables and summaries presented below.

The summary of the measured P and S wave velocities in the boring is as follows:

Boring GEI-7

Depth of Data (feet)	P-wave Velocity (feet/second)	S-wave Velocity (feet/second)	Poisson's Ratio
5 to 20	5225	946	0.4831
20 to 35	5612	1249	0.4739
35 to 51	6574	1476	0.4735
51 to 59.5	9860	3095	0.4454

Poisson's Ratio is calculated as follows:

$$\mu = \frac{V_p^2 - 2V_s^2}{2(V_p^2 - V_s^2)}$$

Where: μ = Poisson's Ratio
Vp = Compression Wave Velocity
Vs = Shear Wave Velocity

The Compression (P) wave energy was generated by a vertical blow to a metal plate placed on the surface, offset from the casing. The zero time of the hammer blow was determined from an impact switch taped to the hammer. Multiple hammer blows were stacked to enhance the downhole energy arrivals.

The Shear (S) Wave energy source was a 6 by 6-inch plank offset from the casing. The long direction of the plank was placed tangent to a circle with the radius center at the borehole. An impact switch taped to the handle of the hammer determined zero time.

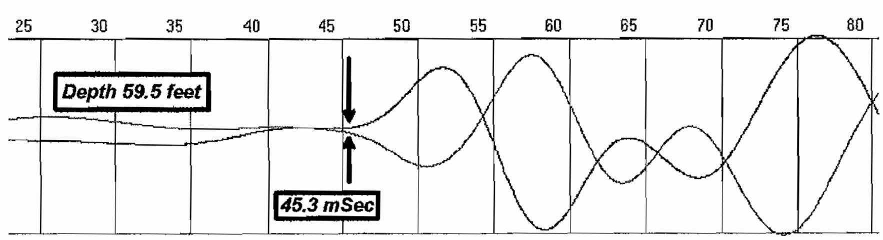
The source offset from the borehole provides a wave travel path that is in soil that is generally not within the soils adjacent to the borehole, that are disturbed by the drilling operation. The disturbed soil zone may be considered to be relatively constant from the top of the boring to the bottom.

The wave motion was detected by a geophone detector lowered in the casing at 5 foot intervals. The detector contains 3, 14 Hz geophones oriented in orthogonal directions. A spring tensioned the detector against the wall of the casing. The orientation of the tri-axial detector package was maintained by a flux gate compass and servo mechanism which rotated the geophone elements so that longitudinal geophone is parallel to the shear wave motion generated on the surface.

For the S wave data, two recordings were made at each data point. The two separate recordings were made with reversed (polarized) energy inputs utilizing the opposite ends of the plank (blows right and left). The time arrival of the shear wave energy was determined by comparing arrival times and direction of particle motion of the recorded wave motion in the two data sets..

The particle motion of the shear wave energy is polarized and is dependent on the direction of the energy input. On Blow 1, the particle motion is reversed from that produced by Blow 2 made on the opposite end of the plank. The polarization of the energy helps the interpreter to separate S wave arrivals from other energy arrivals and noise. Reversed particle motion, however, can also occur in other ways such as out-of-phase noise, shear energy generated in the boring annulus backfill and casing, as tube waves, and P to S conversions. Continuity of the energy arrivals from the surface to the bottom of the hole helps separate these various energy arrivals.

A data sample of the Shear wave data made in Boring GEI-7 at a depth of 59.5 feet is presented below. This data was collected with the Geometrics Geode seismograph. The shear wave arrival is shown on the longitudinal trace. The black trace is the right blow and the blue trace is the left blow.



The picked arrival times were converted from the "slant distance" travel path to the vertical travel path down the borehole. The "slant distance" travel path is a result of the source to borehole offset. The formula used for the conversion to the 'Corrected Time' vertically down the borehole is:

DH Time = Record Time x [Cos(Arctan (offset/detector depth))]

Borehole drift was not measured in the borings, and no corrections have been applied for possible drift. The velocity changes generally correspond to the logged material changes, so that extreme drift of the borings off of vertical is not expected.

The recording equipment was a Geometrics Geode with at sampling rate of 32 micro-seconds and a record length of 200 milli-Seconds, a 12-channel signal enhancement, floating point digital-recording seismograph operated from a laptop computer. Arrival times were picked from a computer screen image of the records.

The information presented in this report is based upon geophysical measurements made by generally accepted methods and field procedures, and our interpretation of these data. The presented information is based upon our best estimate of subsurface conditions considering the geophysical results and all other information available to us. These results are interpretive in nature and are considered to be a reasonably accurate presentation of the existing conditions within the limitations of the method or methods employed.

For Geo-Recon International:

John M Musser Jr

John M. Musser Jr.
Principal Geophysicist

Downhole Compressional and Shear Wave Velocity Measurements

Borehole: GEI-7 - VA Hospital
Seattle, Washington

Shear Wave Data - Interval Velocity Computations

Depth of Data	Recorded Time	Corrected Time	Interval Time	Interval Velocity	Average Velocity
5.0	6.20	3.969	3.969	--------	n/a
10.0	10.70	9.175	5.206	960	
15.0	15.70	14.577	5.402	926	946
20.0	20.70	19.827	5.250	952	
	Velocity Change at ~ 20 feet				
25.0	24.50	23.823	3.996	1251	
30.0	28.30	27.750	3.927	1273	1249
35.0	32.30	31.836	4.085	1224	
	Velocity Change at ~ 35 feet				
40.0	35.70	35.305	3.469	1441	
45.0	39.00	38.658	3.353	1491	1476
50.0	42.30	41.999	3.341	1497	
	Velocity Change at ~ 51 feet				
55.0	43.90	43.641	1.642	3044	
59.5	45.30	45.071	1.430	3146	3095

Bottom of Casing at 61.5 feet.

Source to Borehole offset: 6 feet.
Casing stickup above ground: 0 feet.
n/a - Not included in Velocity Average.

Velocities in feet per second.
Depths in feet - Times in milli-seconds.
Velocity breaks from Time-Depth Plot.

Downhole Compressional and Shear Wave Velocity Measurements

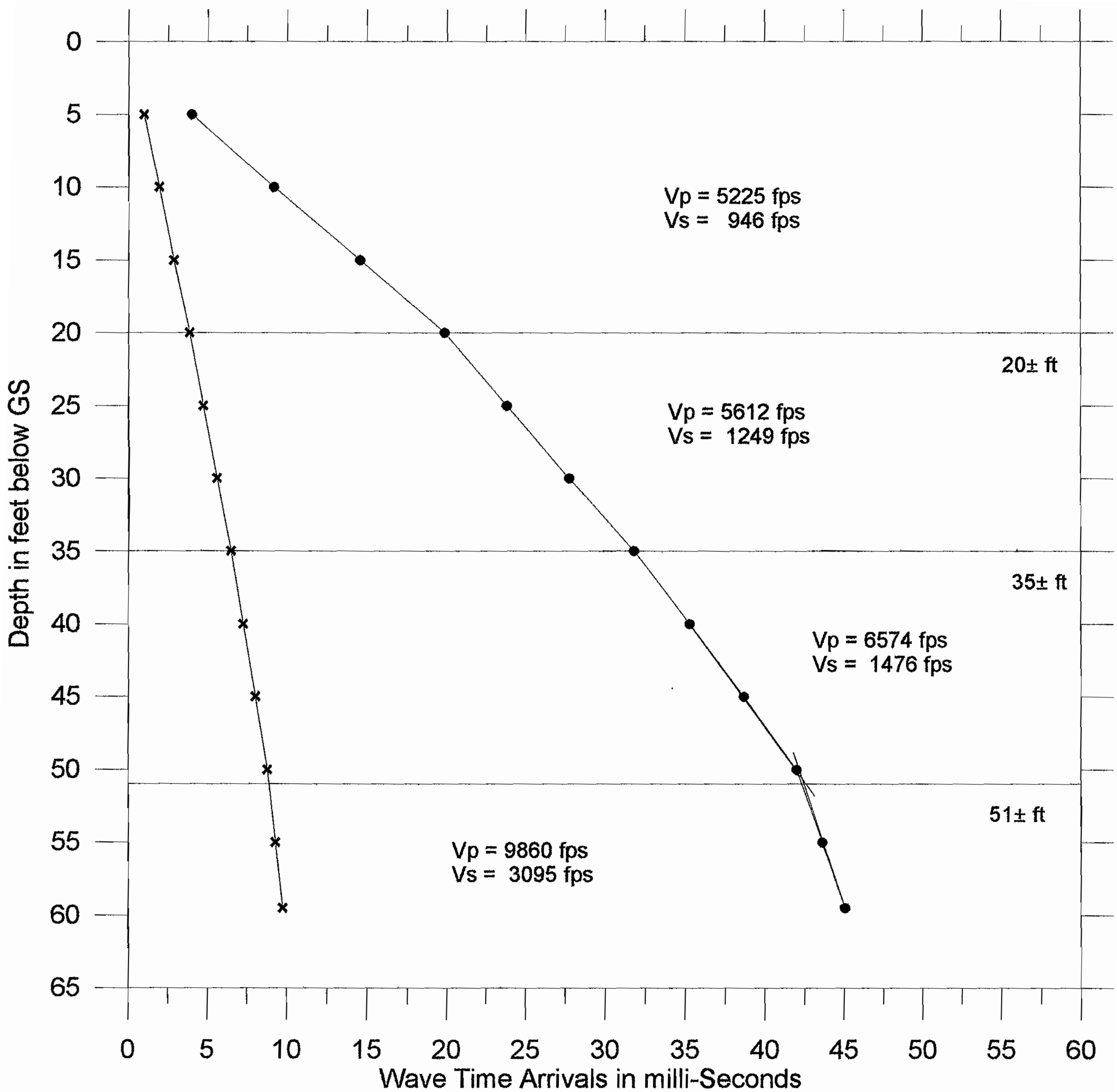
Borehole: GEI-7 - VA Hospital
Seattle, Washington

Compressional Wave Data - Interval Velocity Computations

Depth of Data	Recorded Time	Corrected Time	Interval Time	Interval Velocity		Average Velocity
5.0	1.500	0.960	0.960	-------	n/a	
10.0	2.250	1.929	0.969	5160		
15.0	3.100	2.878	0.949	5269		5225
20.0	4.000	3.831	0.953	5246		
	Velocity Change at ~ 20 feet					
25.0	4.850	4.716	0.885	5651		
30.0	5.700	5.589	0.873	5726	n/a	5612
35.0	6.600	6.505	0.916	5460		
	Velocity Change at ~ 35 feet					
40.0	7.350	7.269	0.764	6548		
45.0	8.100	8.029	0.760	6577		6574
50.0	8.850	8.787	0.758	6596		
	Velocity Change at ~ 51 feet					
55.0	9.350	9.295	0.508	9845		
59.5	9.800	9.751	0.456	9875		9860
	Bottom of Casing at 61.5 feet.					

Source to Borehole offset: 6 feet.
Casing stickup above ground: 0 feet.
n/a - Not included in Velocity Average.

Velocities in feet per second.
Depths in feet - Times in milli-seconds.
Velocity Breaks from Time-Depth Plot.



GEI - 7 VA Hospital - Seattle, Washington
Compression and Shear Wave Velocities

● Shear Wave Arrivals

× Compressional Wave Arrival

Figure 1

APPENDIX D
Previous Explorations

APPENDIX D
PREVIOUS EXPLORATIONS

The reviewed geotechnical information includes:

- The logs of 29 borings completed on the VA campus by Shannon and Wilson, Inc. in 1979, and
- The logs of two borings completed in 2008 by Otto Rosenau & Associates.

OTTO ROSENAU & ASSOCIATES, INC.
6747 M.L. King Way South
Seattle, WA 98118
Telephone: (206) 725-4600
Fax: (206) 723-2221

BORING NUMBER B-1

CLIENT HDR Architecture/Engineering, Inc.
PROJECT NAME VA B100 Emergency Expansion
PROJECT NUMBER 08-0191
PROJECT LOCATION 1660 South Columbian Way, Seattle, WA
DATE STARTED 3/18/08 **COMPLETED** 3/18/08
GROUND ELEVATION 349 ft **HOLE SIZE** 7-inch
DRILLING CONTRACTOR Davies Drilling
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger (Track Rig)
AT TIME OF DRILLING ---
LOGGED BY Craig Bechtold L.G. **CHECKED BY**
AT END OF DRILLING 26.0 ft / Elev 323.0 ft
NOTES 140# safety hammer, SPT sampler, rope and cathead, 30-inch drop
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0	SS 1	67	4-5-6 (11)	MC=14%	SM		Medium dense, brown, Silty SAND with gravel, moist (Fill)
	SS 2	67	10-12-12 (24)	MC=8%	SM		Medium dense, brown, Silty sand with gravel, weathered, moist (Fill)
5	SS 3	67	17-21-50 (71)	MC=12%	SM		Dense, light brown to gray, Silty fine SAND with gravel (Fill)
6.4							342.6
					SM		Very Dense, light brown, Silty SAND with gravel, weathered till, moist
	SS 4	100	15-15-18 (33)	MC=14%	SM		Dense, light brown, Silty SAND with gravel and medium sand interbeds, moist
10	SS 5	100	20-41-50/5"	MC=9%	SM		Very dense, light brown to gray, Silty SAND with gravel, moist
15	SS 6	113	23-35-50/4"	MC=13%	SM		Grades to more gravel and occasional medium sand interbeds
20.0							329.0
20	SS 7	100	18-17-24 (41)	MC=17%	SP-SM		Dense, light brown, very fine SAND grading to medium SAND, moist
25.0							324.0
25	SS 8	100	15-27-42 (69)	MC=24%	SM		Very dense, light brown to gray, Silty fine SAND and trace oxidized sand (moist to wet)
30.0							319.0
30	SS 9	100	16-26-32 (58)	MC=22%	SP-SM		Very dense, gray, fine to medium SAND with silt (wet)
31.5							317.5
							Bottom of hole at 31.5 feet.

GENERAL BH / TP / WELL VA B100 EMERGENCY ROOM.GPJ GINT US.GDT 4/24/08

OTTO ROSENAU & ASSOCIATES, INC.
6747 M.L. King Way South
Seattle, WA 98118
Telephone: (206) 725-4600
Fax: (206) 723-2221

BORING NUMBER B-2

CLIENT HDR Architecture/Engineering, Inc.
PROJECT NAME VA B100 Emergency Expansion
PROJECT NUMBER 08-0191
PROJECT LOCATION 1660 South Columbian Way, Seattle, WA
DATE STARTED 3/18/08 **COMPLETED** 3/18/08
GROUND ELEVATION 347 ft **HOLE SIZE** 7-inch
DRILLING CONTRACTOR Davies Drilling
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger (Track Rig)
AT TIME OF DRILLING ---
LOGGED BY Craig Bechtold L.G. **CHECKED BY**
AT END OF DRILLING ---
NOTES 140# safety hammer, SPT sampler, rope and cathead, 30-inch drop
AFTER DRILLING ---

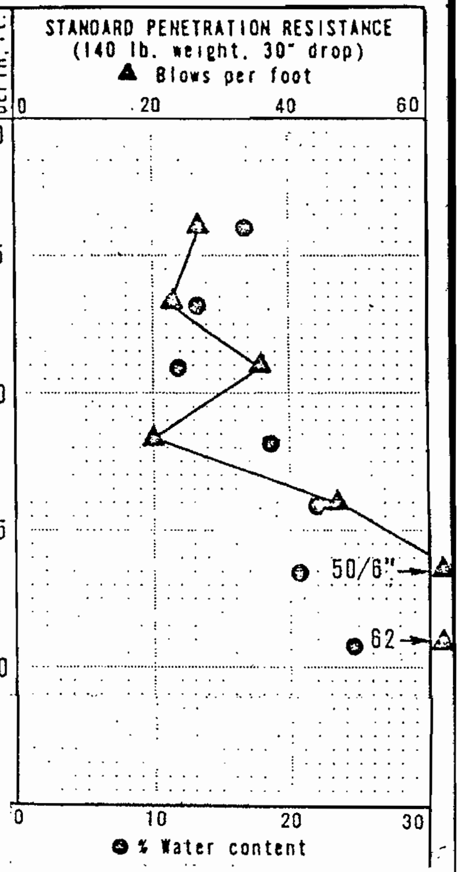
DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0	SS 1	100	5-5-4 (9)	MC=19%	SM		Loose, brown, Silty SAND with gravel, moist (Fill)
	SS 2	100	3-8-4 (12)	MC=16%	SM		Medium dense, gray and brown, Silty fine to medium SAND with gravel (Fill)
5	SS 3	100	2-9-8 (17)	MC=14%	SM		Medium dense, gray, Silty SAND with occasional gravel and charcoal (Fill)
	SS 4	100	8-15-24 (39)	MC=10%	SM		Dense, light brown, Silty fine SAND with gravel (Fill)
10	SS 5	100	12-11-8 (19)	MC=16%	SM		Medium dense, gray, Silty fine SAND with brown oxidized silty sand and charcoal at depth of 10.0' to 11.0' (Fill)
						12.5	334.5
15	SS 6	100	10-12-20 (32)	MC=8%	SP-SM		Medium dense, light brown to brown, fine to medium SAND with silt, moist
20	SS 7	100	13-17-12 (29)	MC=9%	SP-SM		Dense, gray, fine SAND with silt, moist
25	SS 8	100	14-26-35 (61)	MC=8%	SP-SM		Very dense, gray, fine to medium SAND with silt, moist
						26.5	320.5

Bottom of hole at 26.5 feet.

GENERAL BH / TP / WELL VA B100 EMERGENCY ROOM.GPJ GINT US.GDT 4/24/08

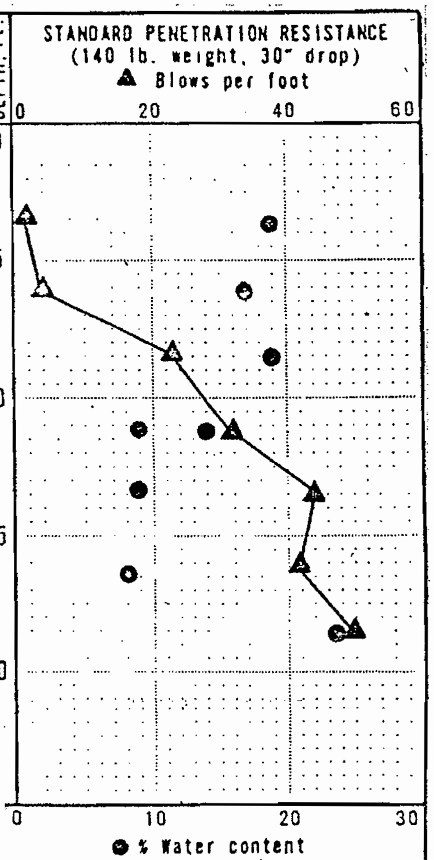
J-1

SOIL DESCRIPTION Surface Elevation: Approx. 308 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, gravelly, sandy SILT.	1		NO GROUNDWATER ENCOUNTERED DURING DRILLING
Very stiff, mottled gray and red-brown, sandy, silty CLAY; trace of gravel and organics (TILL-FILL)	5	1	
Medium dense to dense, clayey, silty SAND; scattered gravel.	12.5	2, 3, 4	
Dense to very dense, silty SAND.	17	5, 6	
Hard, gray, silty CLAY; slightly laminated and thin sandy silt partings.	19.5	7	
BORING COMPLETED 6/4/79			

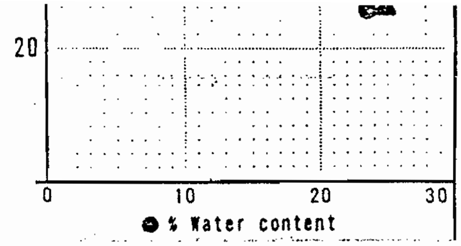


J-2

SOIL DESCRIPTION Surface Elevation: Approx. 312 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Loose, brown, gravelly, sandy SILT; scattered organics (FILL)	0 2.5		
Soft, gray-brown, silty CLAY; occasional organics (FILL)	7.5	1, 2	
Very stiff, gray-brown, silty CLAY; trace of organics.	11	3, 4	
Dense, gray-brown, silty SAND; occasional gravel.	19.0	5, 6, 7	6/4/79
BORING COMPLETED 6/1/79			

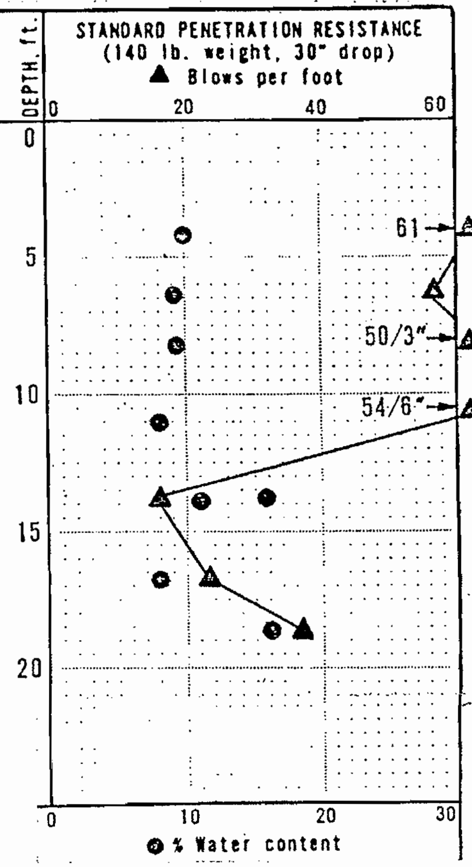


BORING COMPLETED 6/1/79	19.0



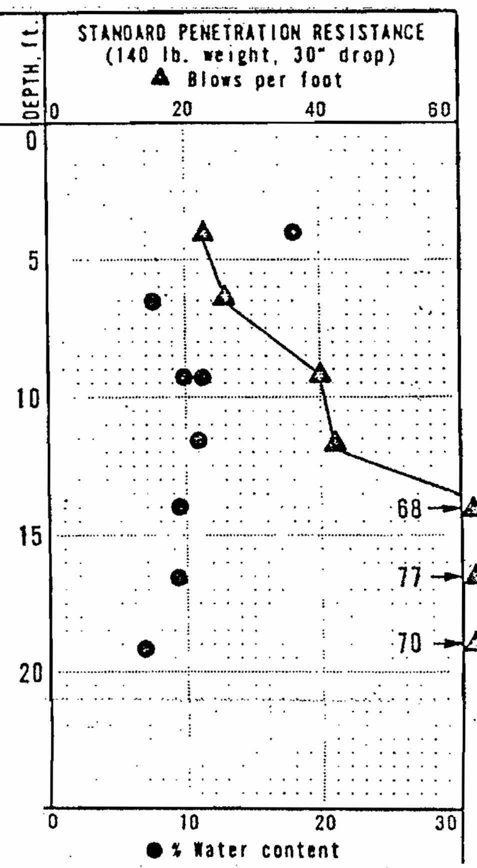
J-3

SOIL DESCRIPTION Surface Elevation: Approx. 315 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Very dense, brown to gray-brown, clayey, silty SAND; trace of gravel.	0	1, 2	NO GROUNDWATER ENCOUNTERED DURING DRILLING
Hard, gray-brown, gravelly, sandy, silty CLAY.	7.5	3	
Very dense, gray-brown, gravelly, clayey, silty SAND.	10	4	
Medium dense to dense, gray-brown to gray, silty to clean SAND; scattered silty layers.	14	5, 6, 7	
BORING COMPLETED 6/1/79	19.5		



J-4

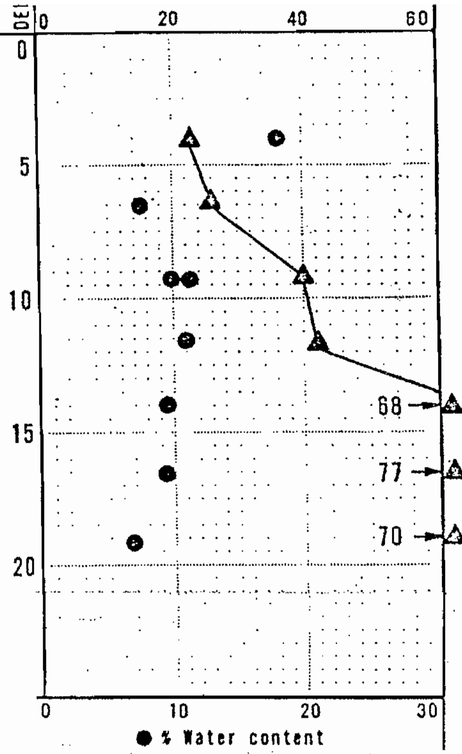
SOIL DESCRIPTION Surface Elevation: Approx. 322 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, slightly gravelly, sandy SILT; numerous roots (FILL)	0		NO GROUNDWATER ENCOUNTERED DURING DRILLING
Very stiff, mottled gray and brown, sandy, silty CLAY; occasional gravel (TILL-FILL)	1.5	1	
Medium dense to dense, gray-brown, silty SAND; occasional gravel.	5	2, 3, 4	
Very dense, gray-brown, slightly silty SAND	12.5	5, 6, 7	
BORING COMPLETED 5/31/79	19.5		



Surface Elevation: Approx. 322 ft.

Soil Description	Depth, ft.	Samples	Ground Water
Brown, slightly gravelly, sandy SILT; numerous roots (FILL)	0 – 1.5		NO GROUNDWATER ENCOUNTERED DURING DRILLING
Very stiff, mottled gray and brown, sandy, silty CLAY; occasional gravel (TILL-FILL)	1.5 – 5	1	
Medium dense to dense, gray-brown, silty SAND; occasional gravel.	5 – 12.5	2, 3, 4	
Very dense, gray-brown, slightly silty SAND	12.5 – 19.5	5, 6, 7	

BORING COMPLETED 5/31/79

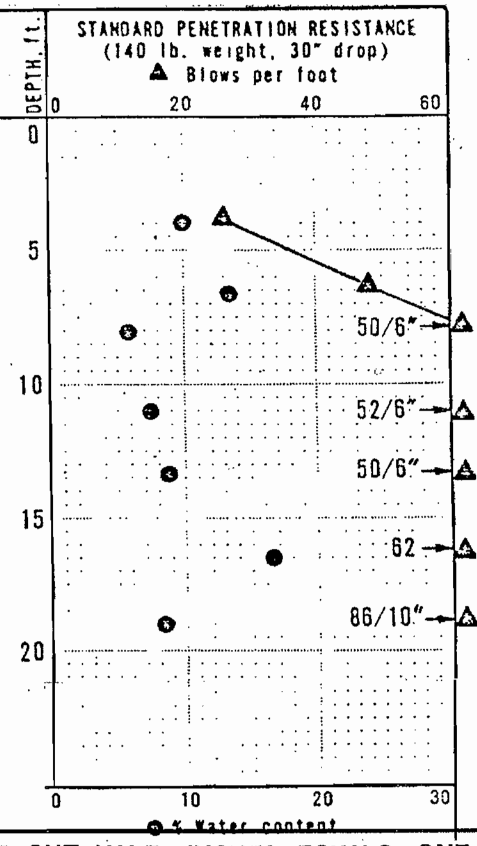


J-5

Surface Elevation: Approx. 339 ft.

SOIL DESCRIPTION	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, sandy, gravelly SILT; numerous roots.	0 – 2		NO GROUNDWATER ENCOUNTERED DURING DRILLING
Medium dense to dense, mottled gray and brown, silty SAND; occasional scattered gravel.	2 – 7.5	1, 2	
Very dense, gray-brown, silty, gravelly SAND; clayey below approx. 17.5 ft. (GLACIAL TILL)	7.5 – 19.5	3, 4, 5, 6, 7	

BORING COMPLETED 5/21/79

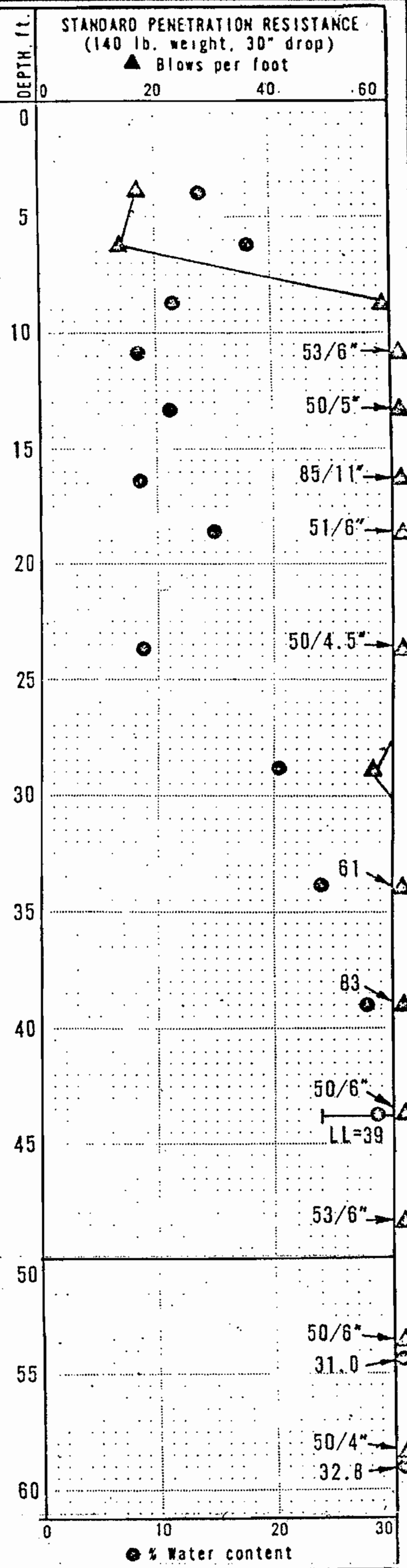


THREE INCHES EQUALS ONE FOOT
0 1

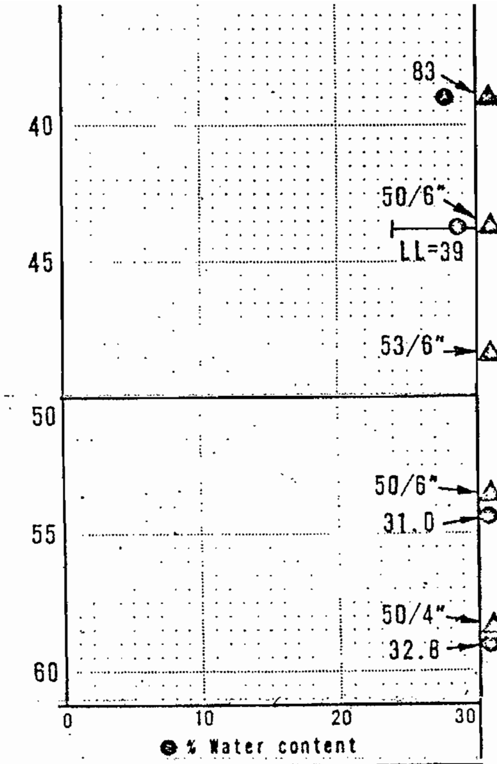
ONE AND ONE HALF INCHES EQUALS ONE F
0 1

-E

SOIL DESCRIPTION Surface Elevation: Approx. 338 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Medium dense, mottled gray-brown, silty, gravelly SAND; numerous roots to 0.5 feet (FILL)	0	1, 2	6/4/79
Very dense, gray-brown, clayey, silty, gravelly SAND (GLACIAL TILL)	7.5	3, 4, 5, 6	
Hard, gray-brown, sandy, silty CLAY; occasional gravel and sand layers.	17.5	7	
Very dense, gray-brown, clean to silty SAND.	21	8, 9, 10	6/4/79
Hard, gray, silty CLAY.	37.5	11, 12	
Very dense, silty, gravelly SAND. TOP OF BEDROCK	44.5		
CLAYSTONE. Very soft, gray, highly broken horizontally bedded.	47	13, 14, 15	
BORING COMPLETED 5/18/79	58.5		

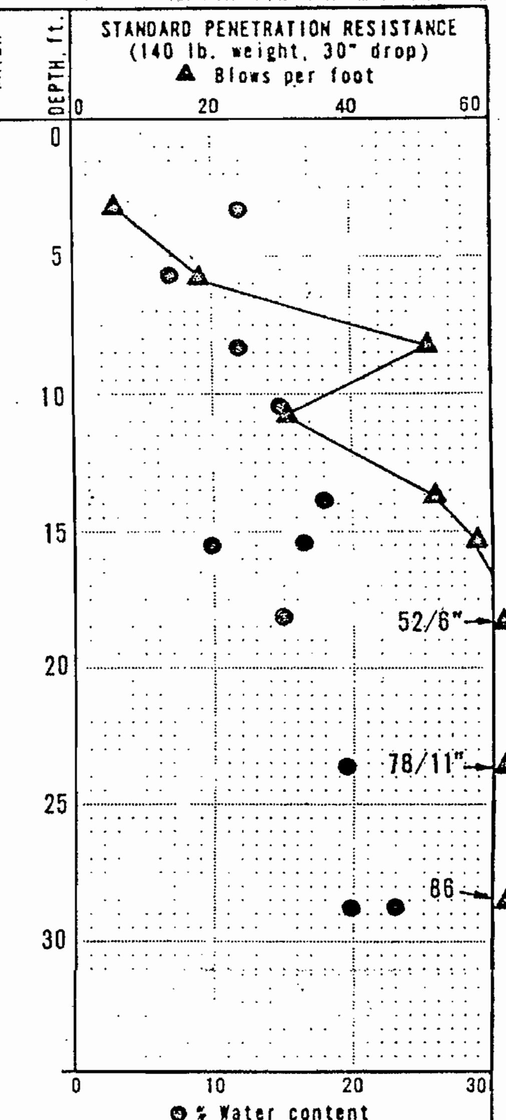


SOIL DESCRIPTION	DEPTH, ft.	SAMPLES
Hard, gray, silty CLAY.	37.5	11, 12
Very dense, silty, gravelly SAND.	44.5	
TOP OF BEDROCK		
CLAYSTONE. Very soft, gray, highly broken horizontally bedded.	47	13, 14, 15
BORING COMPLETED 5/18/79	58.5	



J-7

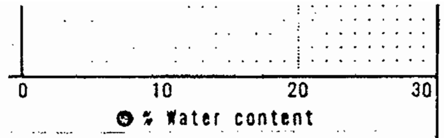
SOIL DESCRIPTION Surface Elevation: Approx. 334 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Loose, brown, silty SAND; scattered gravel and numerous roots to 2 feet (FILL)	0	1	
Medium dense to very dense, silty SAND; layers of sandy SILT and occasional gravel and organics (FILL)	4.5	2	
Dense, brown, and red-brown, silty, clayey SAND and sandy SILT; scattered gravel (TILL-FILL) (FOR 10", BEGAN BOUNCING ON A ROCK AT 11.3 FT.)	8	3, 4	
Very dense, gray-brown, silty SAND; occasional silt layers and fine gravel (GLACIAL TILL)	12	5, 6, 7	5/23/79
Very dense, red-brown, silty SAND.	22	8, 9	
BORING COMPLETED 5/23/79	29.0		



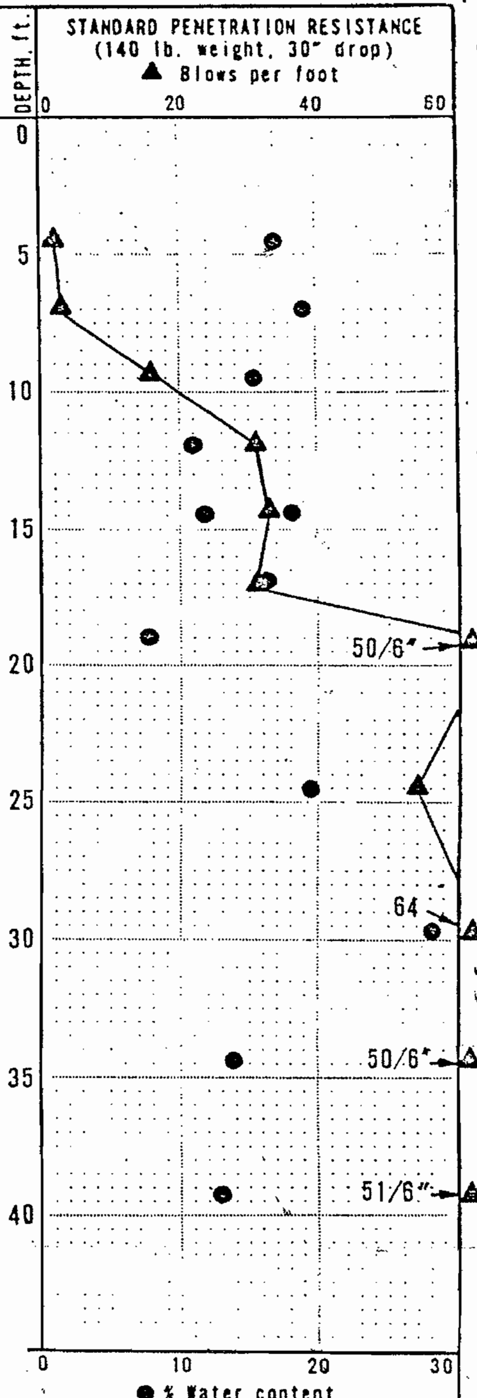
J-8

SOIL DESCRIPTION Surface Elevation: Approx. 333 ft.	DEPTH, ft.	SAMPLES	GROUND WATER	DEPTH, ft.	STANDARD PENETRATION RESISTANCE (140 lb. weight, 30" drop) Blows per foot 0 20 40 60
TOPSOIL	0.0			0	

-1-8



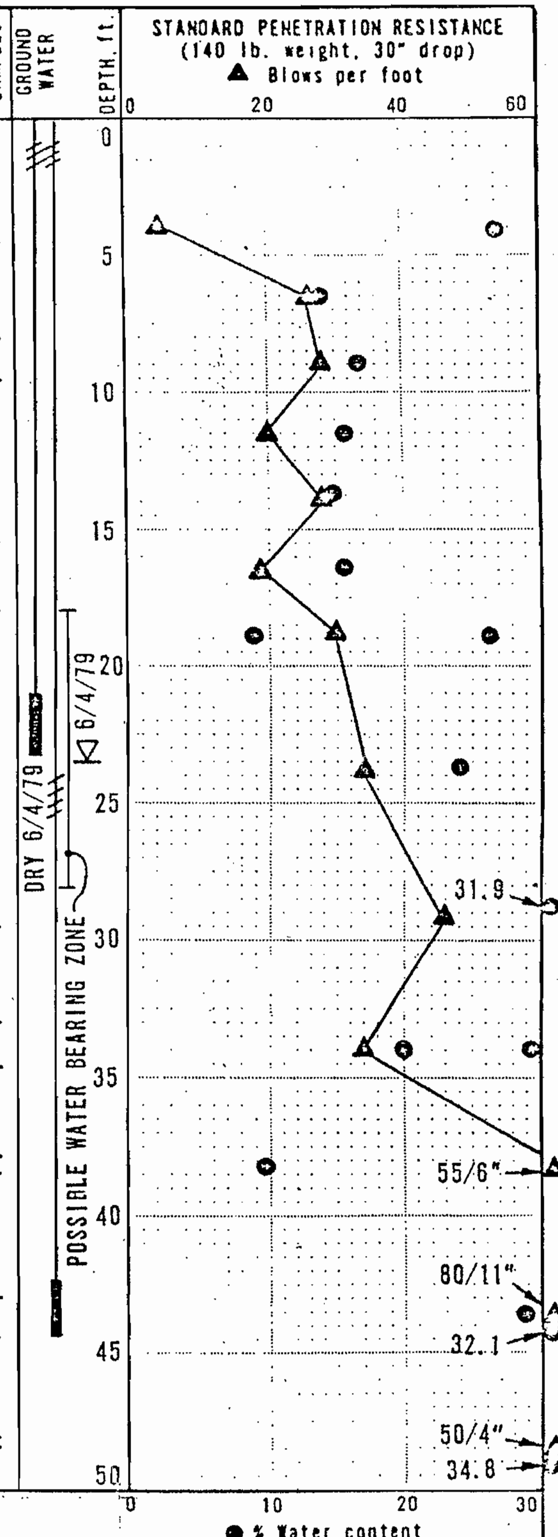
SOIL DESCRIPTION Surface Elevation: Approx. 333 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
TOPSOIL			
Very loose, brown, silty SAND; trace of organics (FILL)	0.9	1, 2	
Medium dense to dense, brown, slightly silty SAND.	8	3, 4	
Hard, gray (red-brown staining) slightly clayey SILT; trace of sand.	14	5, 6	
Very dense, gray-brown, clean to slightly silty SAND.	18	7, 8	
Hard, gray, clayey SILT; occasional fine sand partings.	27	9	
Very dense, gray, sandy, silty CLAY; trace of gravel (GLACIAL TILL)	30.5	10, 11	▽ 5/24/79
BORING COMPLETED 5/24/79	39.5		



FOOT | ONE INCH EQUALS ONE FOOT 0 1 2 | THREE QUARTER INCH EQUALS ONE FOOT 0 1 2 3

J-9

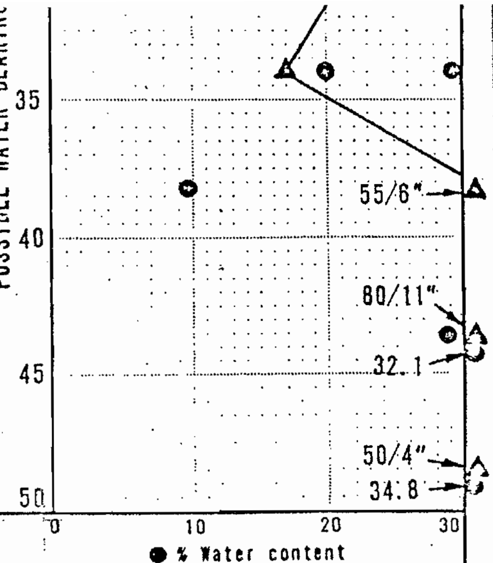
SOIL DESCRIPTION Surface Elevation: Approx. 334 ft.	DEPTH, ft.	SAMPLES
Loose to medium dense, brown to gray-brown, gravelly, clayey, silty SAND.; layers of sandy, clayey SILT, numerous roots, organics and carbonized wood fragments to 5.5 feet (FILL)	0	1, 2, 3
Medium dense, silty SAND; clayey lenses.	10	4, 5, 6
Very stiff, gray-brown, clayey SILT; trace of sand.	17.5	7
Medium dense to dense, gray-brown, clean to slightly silty SAND.	18.5	8
Hard, gray-brown, silty CLAY; occasional sand layers.	28	9, 10
Dense to very dense, gray, sandy, gravelly SILT and silty, gravelly SAND (GLACIAL TILL) TOP OF BEDROCK	37.5	11
SILTSTONE. Soft, gray, highly weathered, highly broken.	43.5	12
BORING COMPLETED 5/21/79	48.3	13



J-10

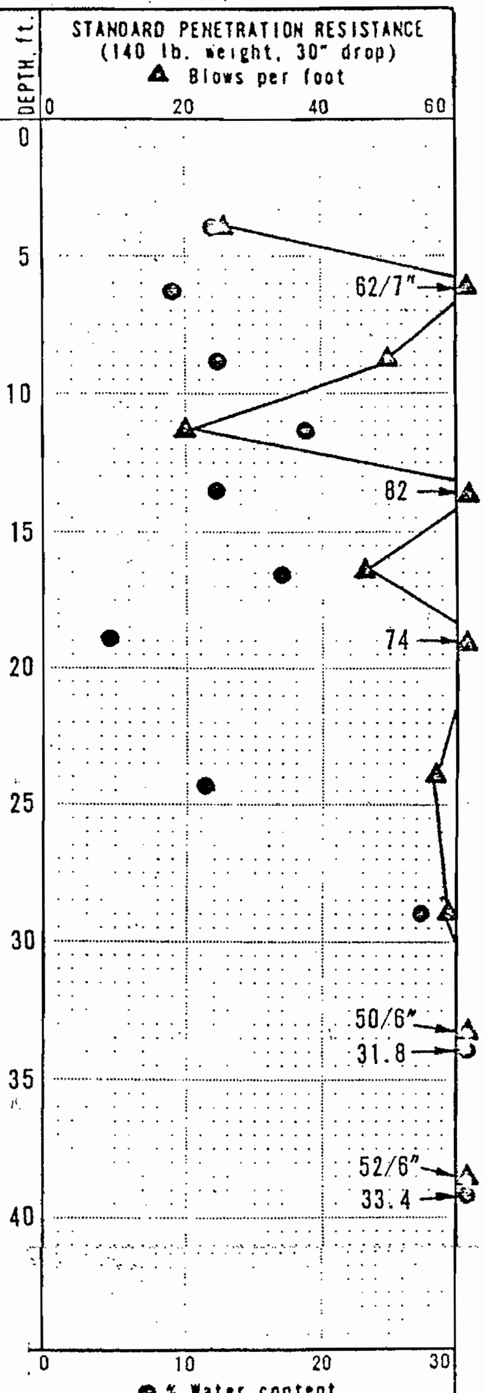
SOIL DESCRIPTION Surface Elevation: Approx. 334 ft.	DEPTH, ft.	SAMPLES	GROUND WATER	DEPTH, ft.	STANDARD PENETRATION RESISTANCE (140 lb. weight, 30" drop) ▲ Blows per foot 0 20 40 60
Brown, gravelly, sandy SILT (rebar at 6 inches and chain link fence wire at 1 foot)(FILL)	0			0	

SOIL DESCRIPTION	DEPTH, ft.	SAMPLES	GROUND WATER
		10	POSSIBLE WATER BEARING
Dense to very dense, gray, sandy, gravelly SILT and silty, gravelly SAND (GLACIAL TILL)	37.5	11	
TOP OF BEDROCK			
SILTSTONE. Soft, gray, highly weathered, highly broken.	43.5	12	
BORING COMPLETED 5/21/79	48.3	13	



J-10

SOIL DESCRIPTION Surface Elevation: Approx. 334 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, gravelly, sandy SILT (rebar at 6 inches and chain link fence wire at 1 foot)(FILL)	0		
Medium dense to very dense, brown, silty SAND; scattered gravel (TILL-FILL)	3	1, 2, 3, 4	
Dense to very dense, gray-brown, silty to clean SAND; occasional thin silt layers.	14	5, 6, 7, 8, 9	5/22/79
TOP OF BEDROCK			
CLAYSTONE. Soft, gray, highly broken.	31	10	
SILTSTONE. Soft, dark gray, highly broken.	35		
BORING COMPLETED 5/22/79	38.5	11	



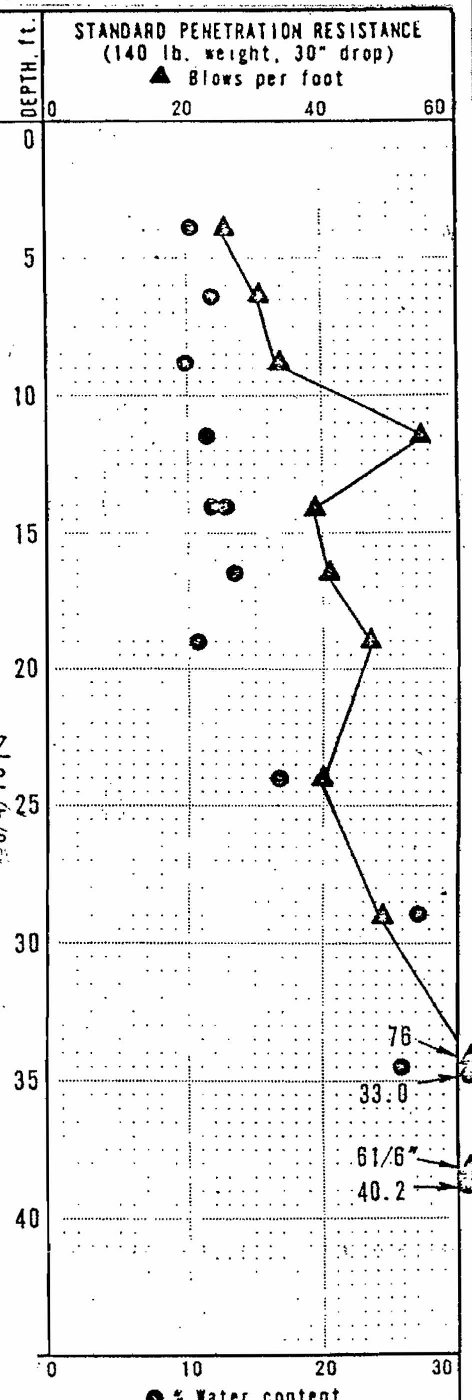
J-11

SOIL DESCRIPTION	ft.	ES	GROUND WATER	ft.	STANDARD PENETRATION RESISTANCE

I-11

SOIL DESCRIPTION Surface Elevation: Approx. 333 ft.	DEPTH, ft.	SAMPLES
Medium dense to dense, gray-brown, clayey, silty, gravelly SAND (FILL)	0	1, 2, 3
Dense to very dense, gray-brown, clayey, silty SAND; scattered gravel.	10	4
Dense, gray-brown, clean to silty SAND; occasional silt layers and gravel.	13.5	5, 6, 7, 8
Hard, gray, silty CLAY; occasional thin fine sand partings. NOTE: Very dense gravelly SAND layer 34.2-34.5 ft. TOP OF BEDROCK	27	9, 10
SILTSTONE. Soft, dark gray, highly weathered, horizontally bedded.	34.5	11
BORING COMPLETED 5/22/79	38.5	

GROUND WATER: 6/4/79; 6/4/79



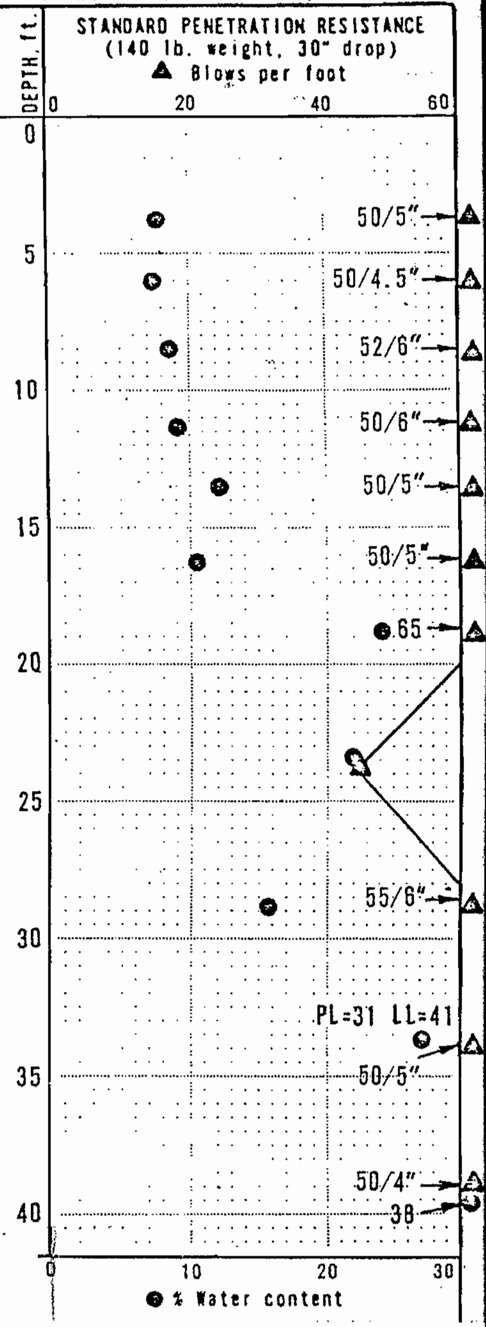
ONE HALF INCH EQUALS ONE FOOT 0 1 2 3 4 5

THREE EIGHTH INCH EQUALS ONE FOOT 0 1 2 3 4 5 6 7 8

ONE 1 2 3

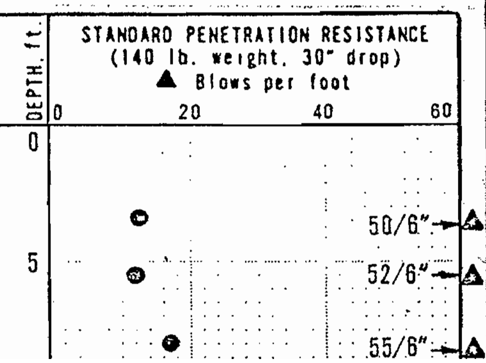
J-12

SOIL DESCRIPTION Surface Elevation: Approx. 333 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, gravelly, sandy SILT.			
Very dense, gray-brown, gravelly, clayey, silty SAND; layers of clayey SILT (GLACIAL TILL)	1	1, 2, 3, 4, 5, 6	
Hard, gray-brown, clayey SILT; trace of sand.	17	7	
Dense to very dense, gray-brown, slightly silty to clean SAND.	21	8, 9	▽ 5/29/79
Hard, gray, clayey SILT; lenses and laminations of silty CLAY.	31	10	
TOP OF BEDROCK			
SILTSTONE; gray, weathered, fractured.	36.5	11	
BORING COMPLETED 5/29/79	38.8		

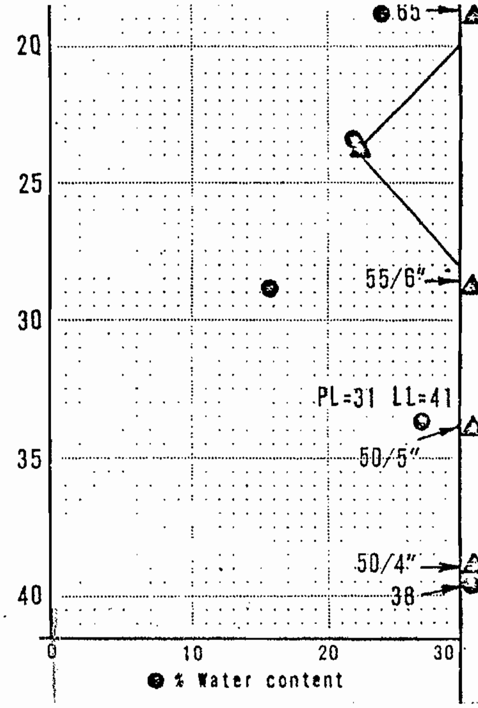


J-13

SOIL DESCRIPTION Surface Elevation: Approx. 342 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, gravelly, sandy SILT.	0		...D DURING DRILLING
Very dense, gray-brown, silty, gravelly SAND; occasional clayey lenses. Gray below 15 feet. (GLACIAL TILL)	2.5	1, 2, 3	

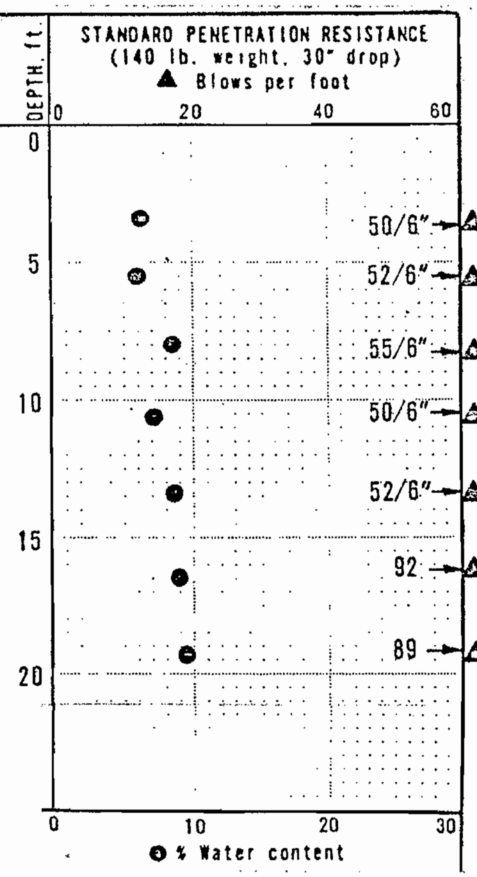


Soil Description	Depth, ft.	Samples	Ground Water
		7	
Dense to very dense, gray-brown, slightly silty to clean SAND.	21	8, 9	▽ 5/29/79
Hard, gray, clayey SILT; lenses and laminations of silty CLAY.	31	10	
TOP OF BEDROCK			
SILTSTONE; gray, weathered, fractured.	36.5	11	
BORING COMPLETED 5/29/79	38.8		



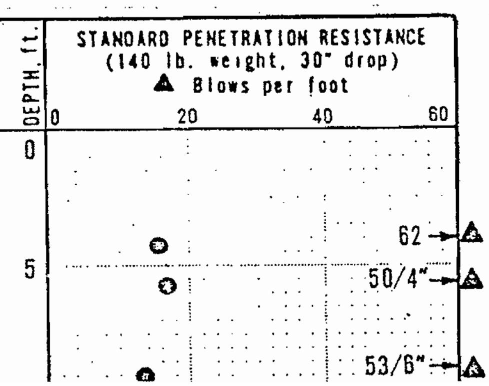
J-13

SOIL DESCRIPTION Surface Elevation: Approx. 342 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, gravelly, sandy SILT.	0		NO GROUNDWATER ENCOUNTERED DURING DRILLING
Very dense, gray-brown, silty, gravelly SAND; occasional clayey lenses. Gray below 15 feet. (GLACIAL TILL)	2.5	1, 2, 3, 4, 5, 6, 7	
BORING COMPLETED 5/22/79	19.5		



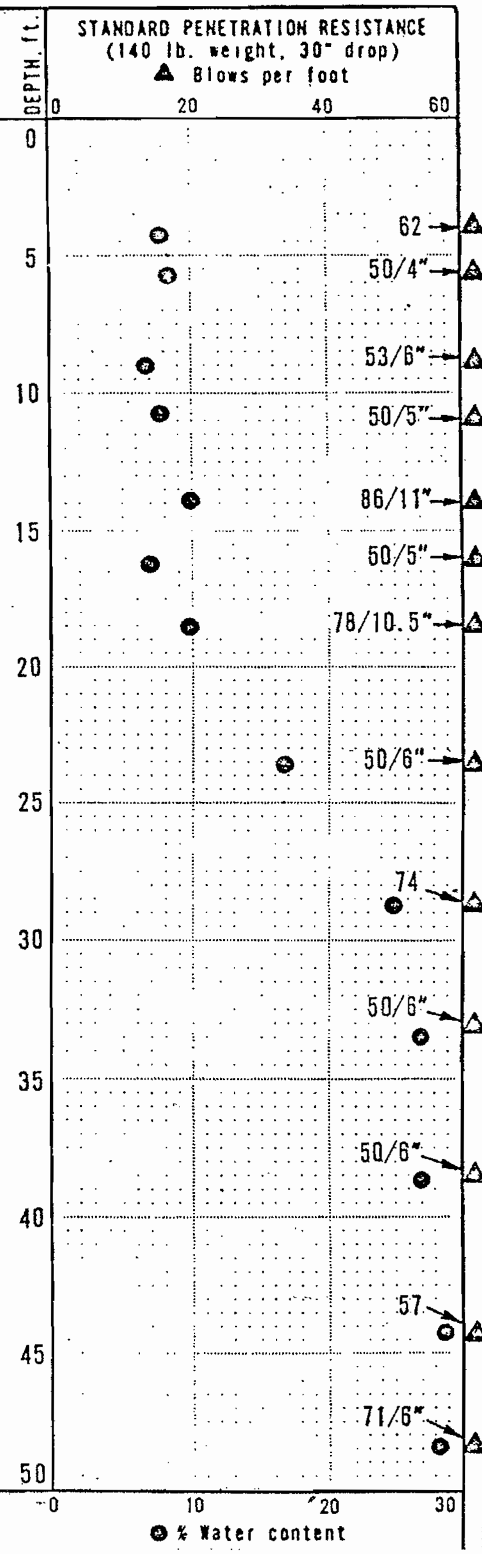
J-14

SOIL DESCRIPTION Surface Elevation: Approx. 337 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, silty, fine SAND; occasional scattered gravel.	0		
Very dense, gray-brown, silty, gravelly SAND; grading locally to gravelly, sandy SILT (GLACIAL TILL)	2.5	1, 2, 3	



J-14

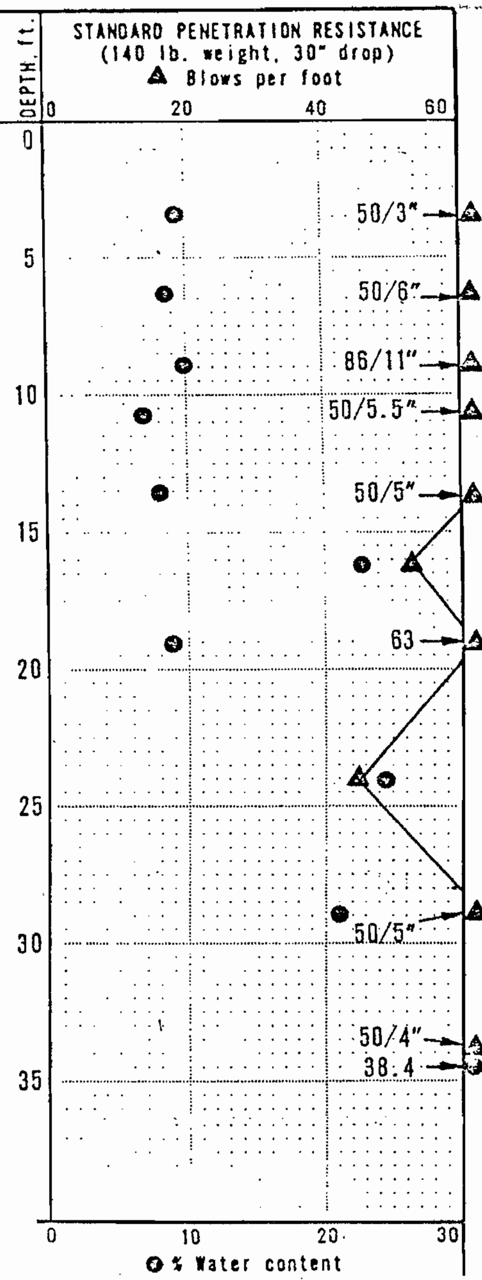
SOIL DESCRIPTION Surface Elevation: Approx. 337 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, silty, fine SAND; occasional scattered gravel.	0		
Very dense, gray-brown, silty, gravelly SAND; grading locally to gravelly, sandy SILT (GLACIAL TILL)	2.5	1, 2, 3, 4, 5, 6, 7	
Hard, gray-brown, clayey SILT; occasional layers of silty sand and gravel.	23	8	6/4/79
Very dense, gray, clean to slightly silty SAND.	27	9	
Hard, gray, clayey SILT. TOP OF BEDROCK	33	10, 11, 12	
SILTSTONE: Very soft to soft, dark gray, highly broken, highly weathered.	45	13	
BORING COMPLETED 5/18/79	48.5		



6-3-1-0530

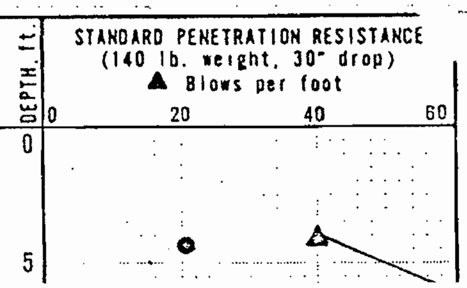
J-15

SOIL DESCRIPTION Surface Elevation: Approx. 332 ft.	DEPTH, ft	SAMPLES	GROUND WATER
Very dense, brown and gray-brown, silty SAND; scattered gravel (GLACIAL TILL)	0	1, 2, 3, 4, 5	
Hard, brown, sandy SILT; occasional thin fine sand partings.	14.5	6	
Very dense, gray-brown, clean to slightly silty SAND. TOP OF BEDROCK	17	7, 8, 9	6/4/79
SILTSTONE. Soft, gray, highly broken.	29.5	10	
BORING COMPLETED 5/23/79	33.8		

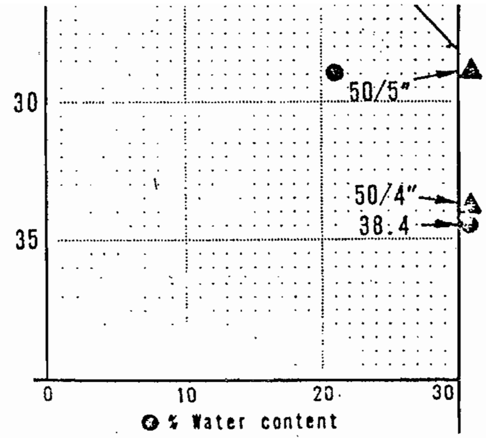


J-16

SOIL DESCRIPTION Surface Elevation: Approx. 333 ft.	DEPTH, ft	SAMPLES	GROUND WATER
Brown, slightly gravelly, sandy SILT; numerous roots.	0		
Dense to very dense, gray-brown, clayey, silty SAND (GLACIAL TILL)	2.5	1	

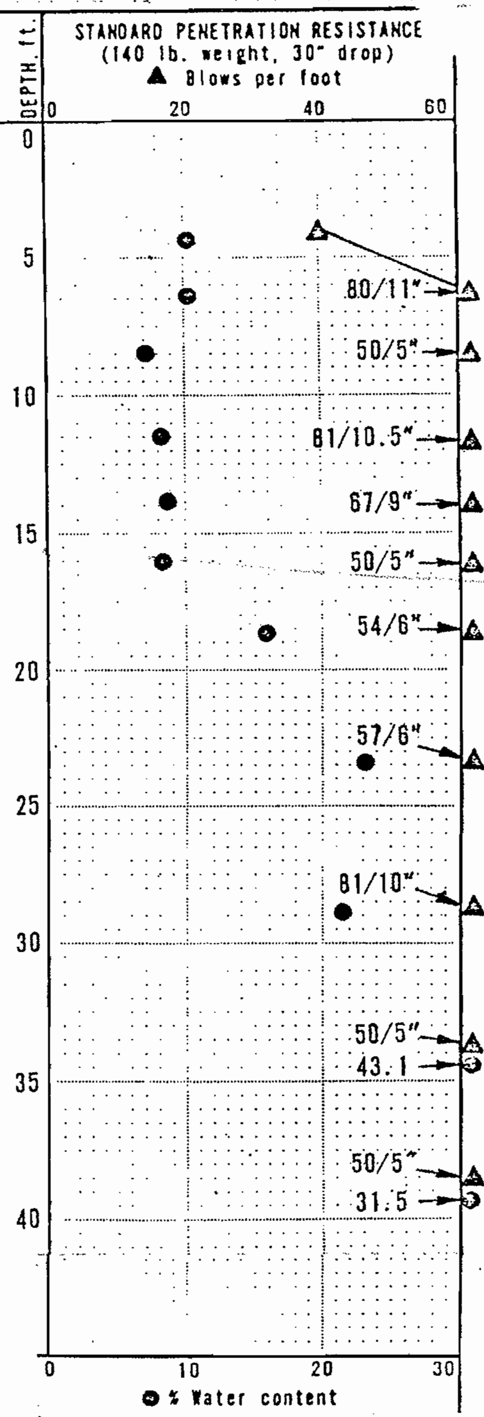


SOIL DESCRIPTION	DEPTH, ft.	SAMPLES
TOP OF BEDROCK		9
SILTSTONE. Soft, gray, highly broken.	29.5	10
BORING COMPLETED 5/23/79	33.8	



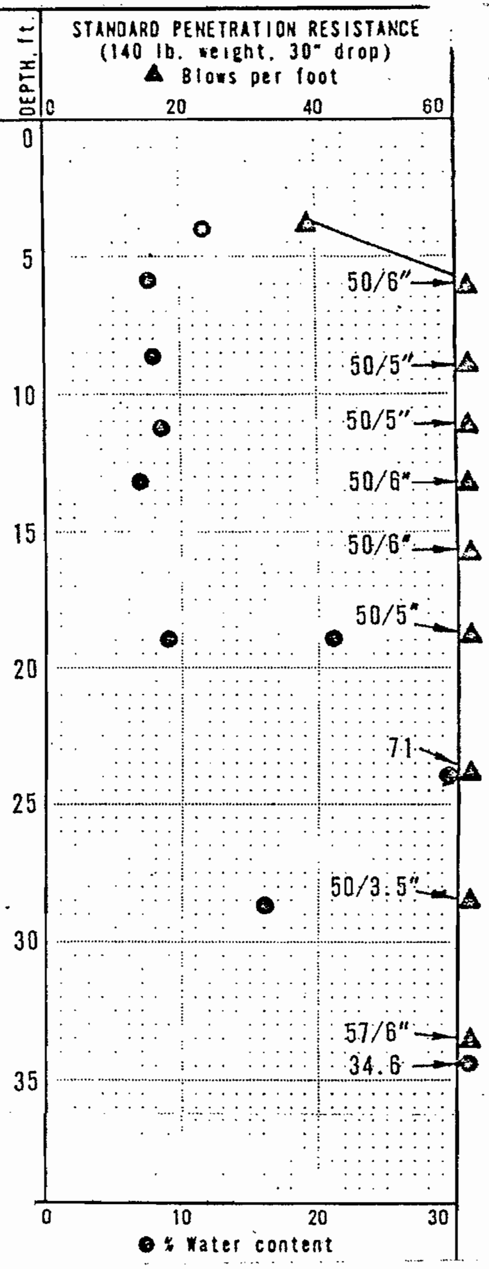
J-16

SOIL DESCRIPTION Surface Elevation: Approx. 333 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, slightly gravelly, sandy SILT; numerous roots.	0		
Dense to very dense, gray-brown, clayey, silty SAND (GLACIAL TILL)	2.5	1, 2, 3, 4, 5, 6, 7	
Very dense, gray, clean to silty SAND.	20	8, 9	5/31/79
TOP OF BEDROCK			
SILTSTONE. Soft, gray, highly broken, highly weathered.	30	10, 11	
BORING COMPLETED 5/31/79	38.4		



J-17

SOIL DESCRIPTION Surface Elevation: Approx. 331 ft.	DEPTH, ft	SAMPLES	GROUND WATER
Dense to very dense, gray-brown, gravelly, silty to clayey SAND; occasional clayey silt lenses (GLACIAL TILL)	0	1, 2, 3, 4, 5, 6	
Hard, gray, silty CLAY; numerous thin, fine sand partings.	18.5	7	
Very dense, gray, clean SAND. NOTE: Clayey and gravelly from 28 to 29.5 ft. (GLACIAL TILL) TOP OF BEDROCK	19.5	8, 9	5/24/79
SILTSTONE. Soft, dark gray, highly broken, highly weathered.	29.5		
BORING COMPLETED 5/24/79	33.5	10	

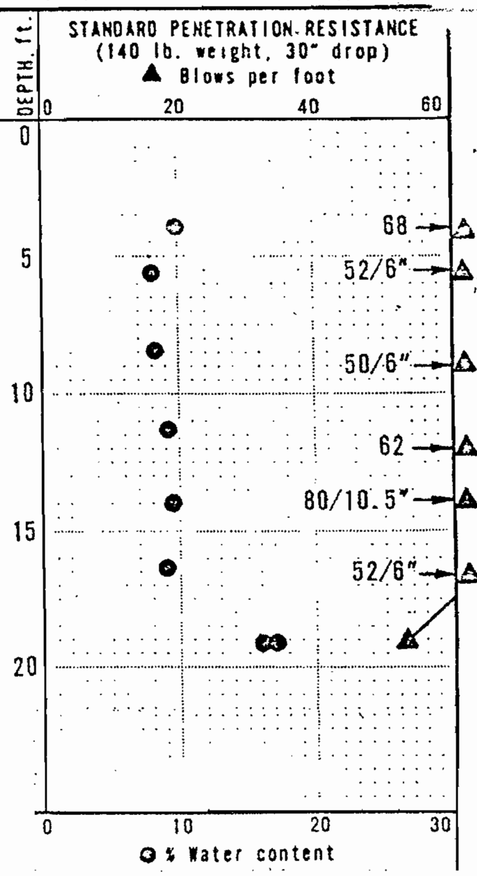


THREE INCHES EQUALS ONE FOOT 0 1

ONE AND ONE HALF INCHES EQUALS ONE FO 0 1

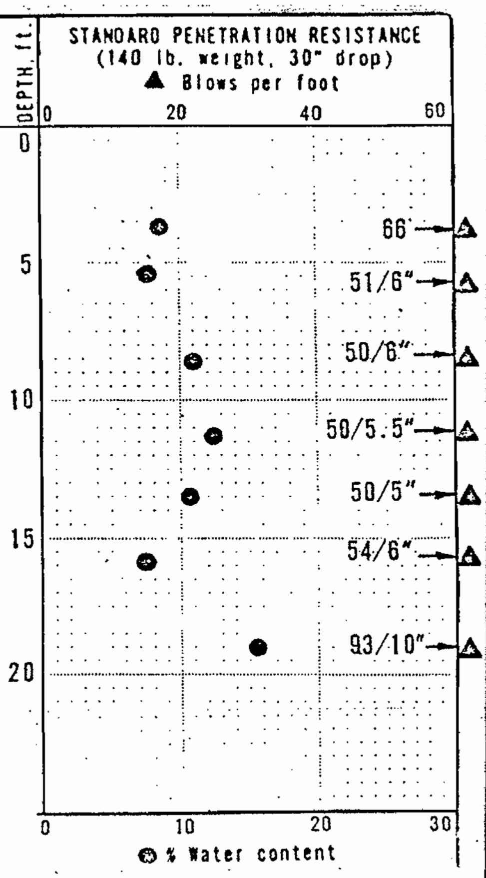
-18

SOIL DESCRIPTION Surface Elevation: Approx. 335 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, gravelly, sandy SILT; numerous roots.	0		
Very dense, gray-brown, silty SAND; scattered gravel (GLACIAL TILL)	2	1, 2, 3, 4, 5, 6	▽ 5/22/79
Dense, gray, clean to slightly silty SAND; with silt layers.	17.5	7	
	19.5		
BORING COMPLETED 5/22/79			



J-19

SOIL DESCRIPTION Surface Elevation: Approx. 329 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, gravelly, sandy SILT; numerous roots.	0		NO GROUNDWATER ENCOUNTERED DURING DRILLING
Very dense, gray-brown, gravelly, clayey, silty SAND (GLACIAL TILL)	1.5	1, 2, 3, 4, 5, 6	
Hard, gray, sandy, silty CLAY; occasional gravel.	18	7	
	19.3		
BORING COMPLETED 5/30/79			

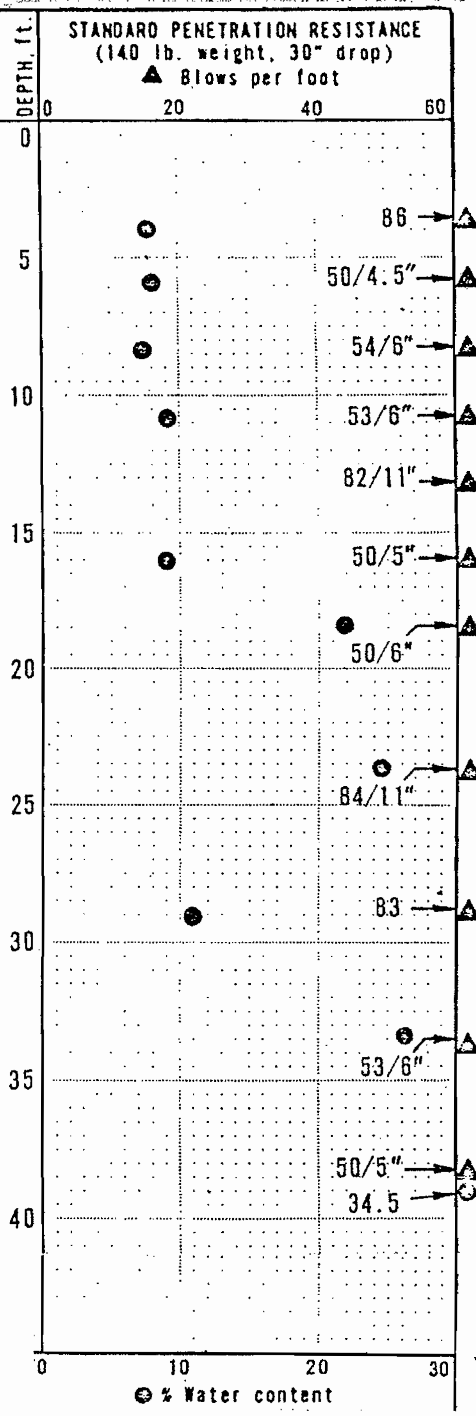


J-20

SOIL DESCRIPTION (Surface Elevation: Approx. 329 ft.)	DEPTH, ft.
Brown, sandy SILT; abundant roots and scattered gravel.	0
Very dense, gray-brown, gravelly, clayey, silty SAND (GLACIAL TILL)	2.5
Hard, gray, sandy, silty CLAY; trace of gravel.	12
Very dense, gray, sandy SILT.	17
Hard, gray, silty CLAY; slightly laminated, with silty sand partings.	20
Hard, gray, sandy, silty CLAY; trace of gravel (GLACIAL TILL)	25
TOP OF BEDROCK	
SILTSTONE; Soft, gray, highly weathered.	31
BORING COMPLETED 5/29/79	38.5

SAMPLES: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11

GROUND WATER: 6/4/79

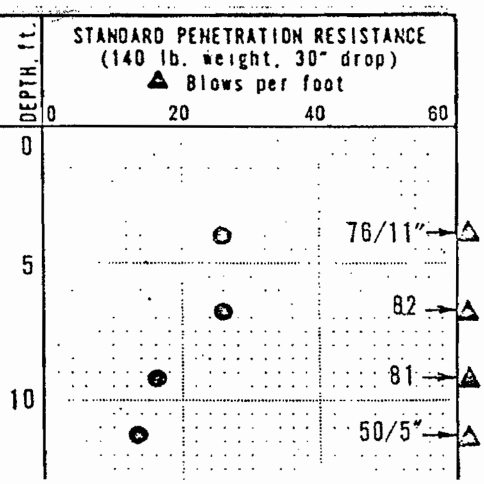


J-21

SOIL DESCRIPTION (Surface Elevation: Approx. 326 ft.)	DEPTH, ft.
Brown, gravelly, sandy SILT; numerous roots.	
Very dense, gray-brown, silty SAND; occasional gravel and clayey lenses and a trace of organics (FILL)	1
Very dense, gray-brown to gray, gravelly, clayey, silty SAND (GLACIAL TILL)	5

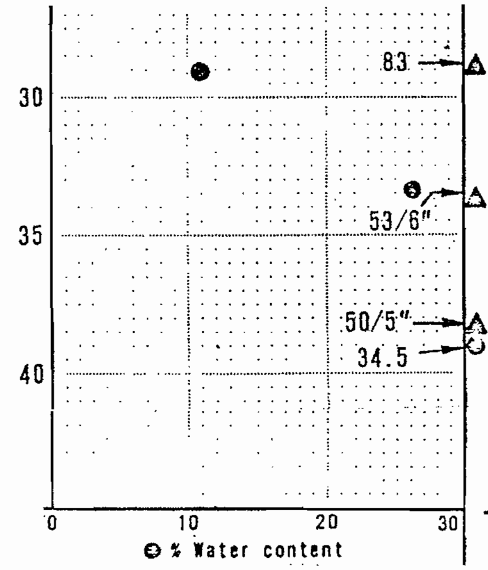
SAMPLES: 1, 2, 3, 4

GROUND WATER: ENCOUNTERED DURING DRILLING



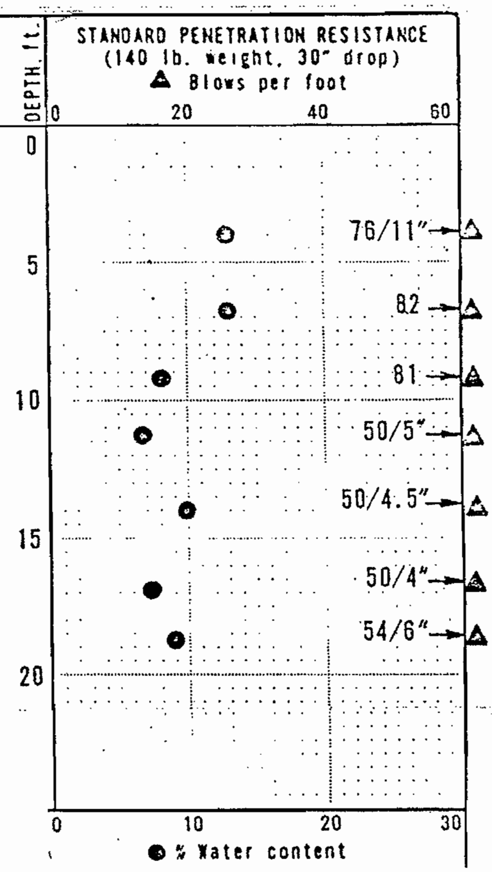
(GLACIAL TILL)

Soil Description	Depth, ft.	Samples
		9
TOP OF BEDROCK	31	
SILTSTONE; Soft, gray, highly weathered.		10
BORING COMPLETED 5/29/79	38.5	11



J-21

SOIL DESCRIPTION Surface Elevation: Approx. 326 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, gravelly, sandy SILT; numerous roots.	1		NO GROUNDWATER ENCOUNTERED DURING DRILLING
Very dense, gray-brown, silty SAND; occasional gravel and clayey lenses and a trace of organics (FILL)	5	1	
Very dense, gray-brown to gray, gravelly, clayey, silty SAND (GLACIAL TILL)		2, 3, 4, 5, 6	
BORING COMPLETED 5/30/79	18.5	7	

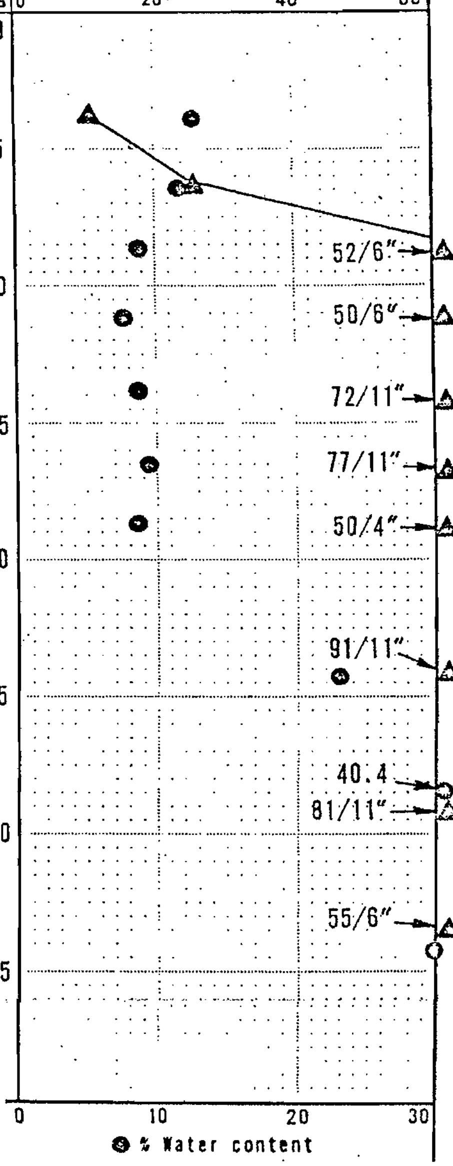


FOOT	ONE INCH EQUALS ONE FOOT	THREE QUARTER INCH EQUALS ONE FOOT
	0 1 2	0 1 2 3

J-22

SOIL DESCRIPTION Surface Elevation: Approx. 330 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Medium dense, gray-brown, gravelly, silty SAND; trace of organics (FILL)	0	1, 2	NO GROUNDWATER ENCOUNTERED DURING DRILLING
Very dense, gray, gravelly, silty SAND; grading locally to hard, sandy CLAY (GLACIAL TILL) NOTE: Hard, silty clay layer 23-24 feet. TOP OF BEDROCK	7.5	3, 4, 5, 6, 7, 8	
SILTSTONE. Soft, gray, highly weathered and broken.	24	9	
BORING COMPLETED 5/24/79	33.5	10	

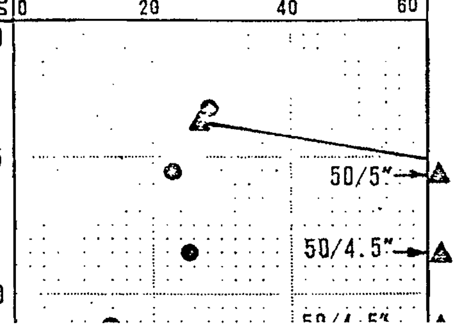
STANDARD PENETRATION RESISTANCE (140 lb. weight, 30" drop)



J-23

SOIL DESCRIPTION Surface Elevation: Approx. 323 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, slightly gravelly, sandy SILT.			PERCHED WATER AT 7 FT.
Medium dense, brown, clayey, silty SAND; occasional gravel.	1	1	
Very dense, gray-brown, slightly gravelly, clayey, silty SAND; scattered gravel (GLACIAL TILL)	5	2, 3	
Very dense, gray, gravelly, silty SAND (GLACIAL	10	4	

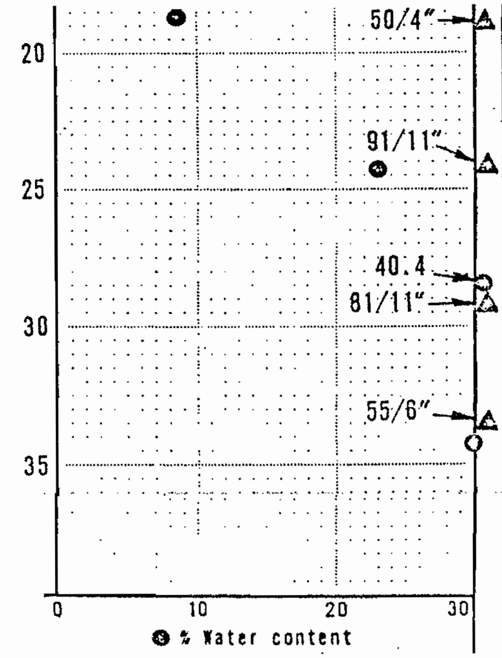
STANDARD PENETRATION RESISTANCE (140 lb. weight, 30" drop)



NOTE: Hard, silty clay layer 23-24 feet.

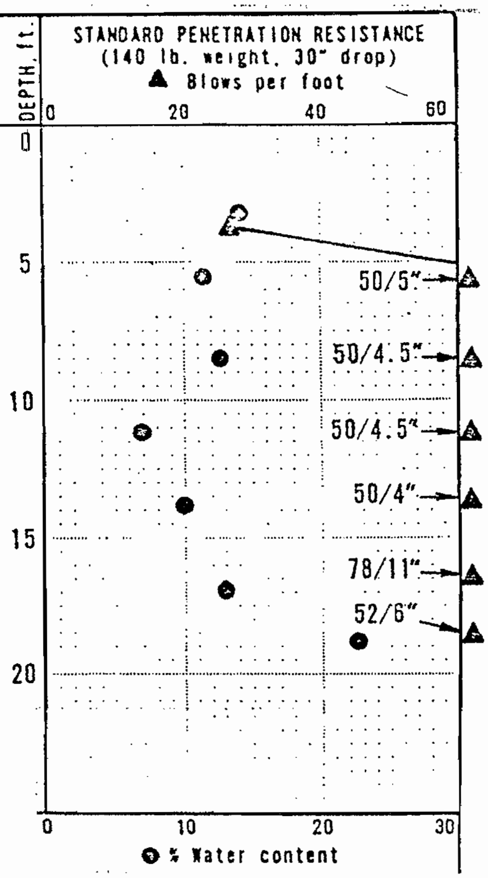
TOP OF BEDROCK

Soil Description	Depth, ft.	Samples	Ground Water
SILTSTONE. Soft, gray, highly weathered and broken.	24	8, 9	NO GROUNDWATER ENC
BORING COMPLETED 5/24/79	33.5	10	



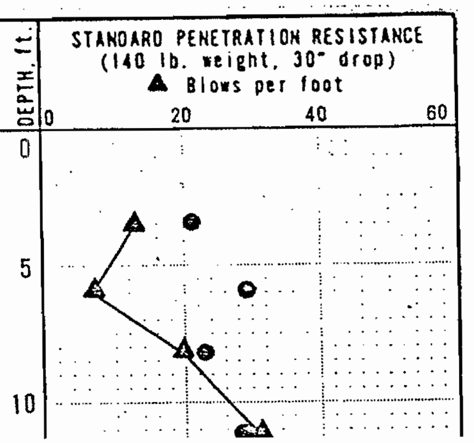
J-23

SOIL DESCRIPTION Surface Elevation: Approx. 323 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, slightly gravelly, sandy SILT.	1		POSSIBLE PERCHED WATER AT 7 FT.
Medium dense, brown, clayey, silty SAND; occasional gravel.	5	1	
Very dense, gray-brown, slightly gravelly, clayey, silty SAND; scattered gravel (GLACIAL TILL)	10	2, 3	
Very dense, gray, gravelly, silty SAND (GLACIAL TILL)	19.0	4, 5, 6, 7	
BORING COMPLETED 5/30/79			

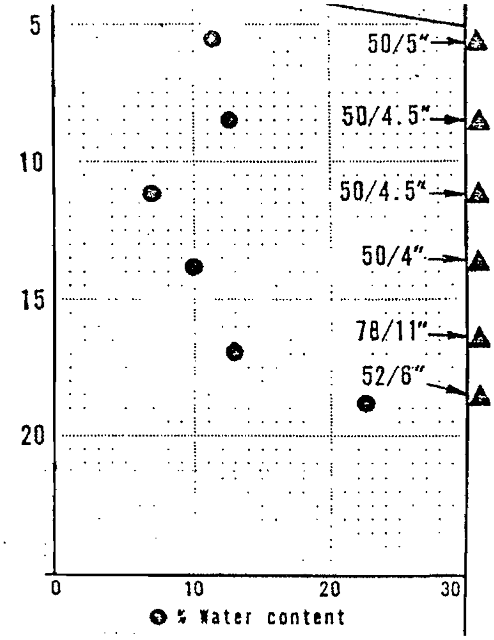


J-24

SOIL DESCRIPTION Surface Elevation: Approx. 331 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Loose to dense, gray-brown, occasionally mottled, silty SAND (TILL-FILL)	0	1, 2, 3, 4	

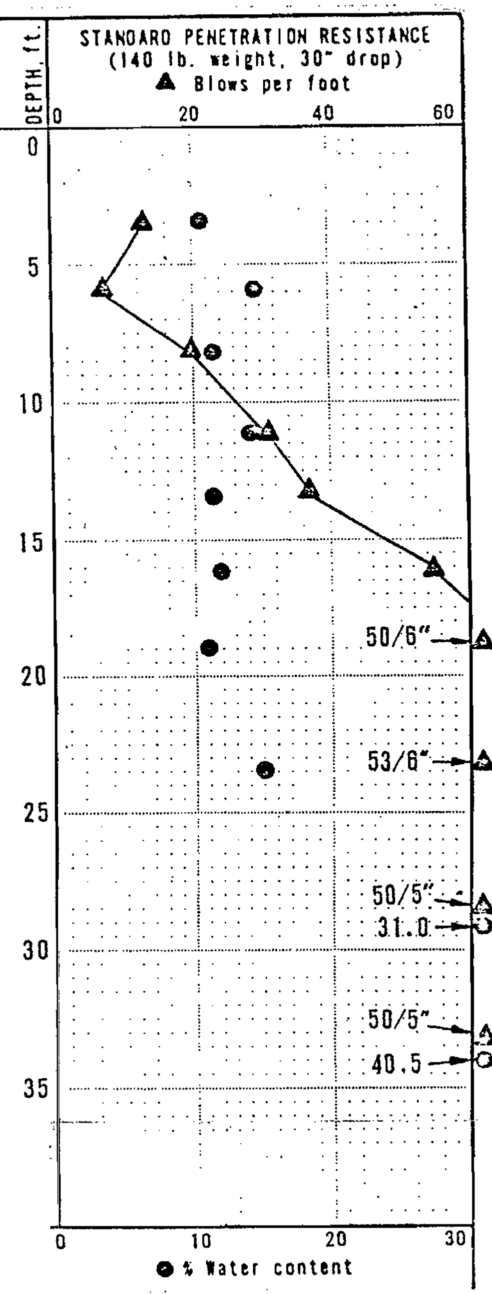


Soil Description	Depth, ft.	Samples	Ground Water
Very dense, gray-brown, slightly gravelly, clayey, silty SAND; scattered gravel (GLACIAL TILL)	5	2, 3	POSSIBLE PERCHED WATER A
Very dense, gray, gravelly, silty SAND (GLACIAL TILL)	10	4, 5, 6, 7	
BORING COMPLETED 5/30/79	19.0		



J-24

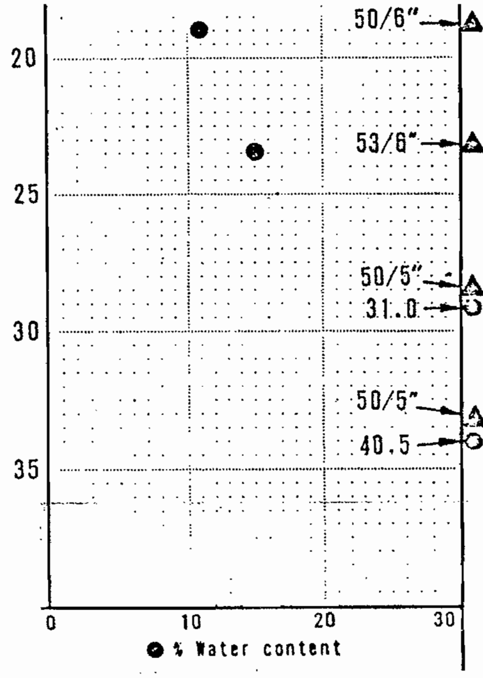
SOIL DESCRIPTION Surface Elevation: Approx. 331 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Loose to dense, gray-brown, occasionally mottled, silty SAND (TILL-FILL)	0	1, 2, 3, 4	
Dense to very dense, gray-brown, clayey, gravelly SAND and hard, sandy CLAY (GLACIAL TILL) TOP OF BEDROCK	12	5, 6, 7, 8	
SILTSTONE. Soft, gray, highly weathered.	28	9, 10	5/31/79
BORING COMPLETED 5/31/79	33.4		



J-25

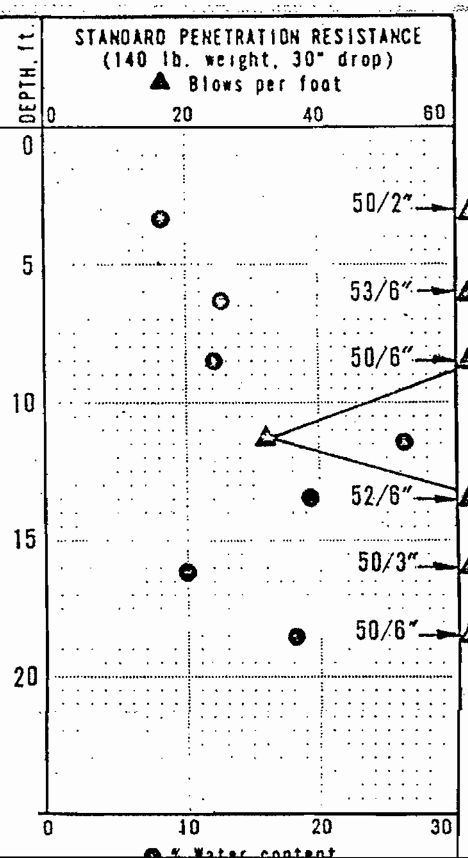
SOIL DESCRIPTION			STANDARD PENETRATION RESISTANCE

Soil Description	Depth, ft.	Samples	Ground Water
		7	
		8	
TOP OF BEDROCK	28		
SILTSTONE. Soft, gray, highly weathered.		9	5/31/79
	33.4	10	
BORING COMPLETED 5/31/79			



J-25

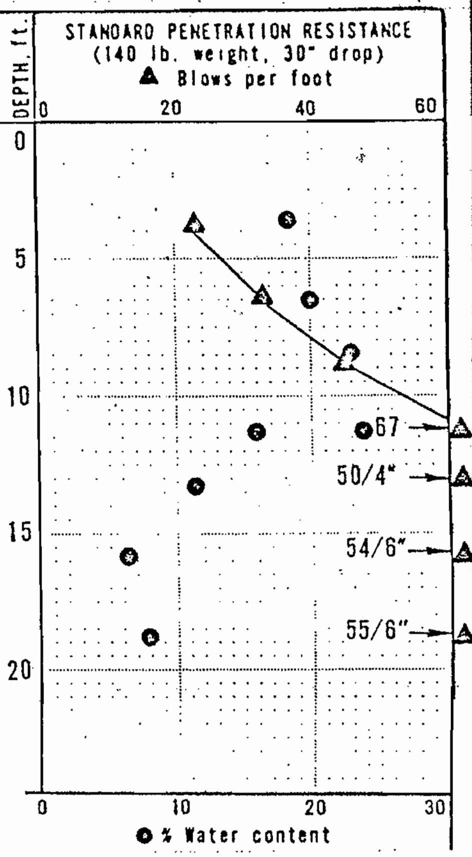
SOIL DESCRIPTION Surface Elevation: Approx. 314 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Very dense, sandy, clayey SILT; trace of gravel and organics (GLACIAL TILL)	0	1	NO GROUNDWATER ENCOUNTERED DURING DRILLING
Very dense, gray-brown, gravelly, clayey, silty SAND.	5	2, 3	
Hard, mottled gray and red-brown, sandy, silty CLAY.	10	4	
Hard, gray-brown, clayey SILT; numerous sand and silt partings.	13	5	
Very dense, gray-brown, gravelly, clayey, silty SAND.	15	6	
Hard, gray, sandy, clayey SILT; trace of gravel (GLACIAL TILL)	17	7	
BORING COMPLETED 6/1/79	19.0		



T ONE HALF INCH EQUALS ONE FOOT 0 1 2 3 4 5
THREE EIGHTH INCH EQUALS ONE FOOT 0 1 2 3 4 5 6 7 8
ONE 1 2 3

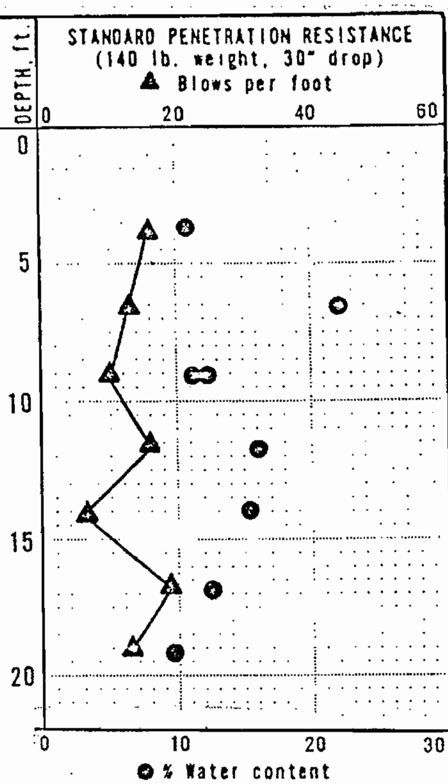
J-26

SOIL DESCRIPTION Surface Elevation: Approx. 313 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Medium dense to dense, gray-brown, silty to slightly silty SAND; occasional clayey silt layers.	0	1, 2	NO GROUNDWATER ENCOUNTERED DURING DRILLING
Hard, gray, silty CLAY; layers to 6 inches of silty to clean sand.	7.5	3, 4	
Very dense, gravelly, clayey, silty SAND (GLACIAL TILL)	12.5	5, 6, 7	
BORING COMPLETED 6/4/79	18.5		



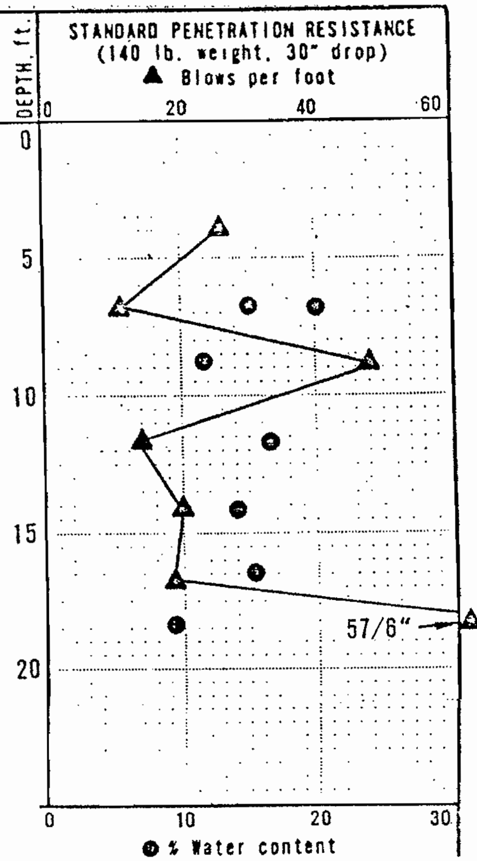
J-27

SOIL DESCRIPTION Surface Elevation: Approx. 309 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Medium dense, gray-brown, gravelly, silty, clayey SAND; trace of organics (TILL-FILL) From 8.5 to 10 feet: Stiff, organic clayey SILT.	0	1, 2, 3, 4	
Loose to medium dense, silty SAND; trace of gravel and clay.	12.5	5, 6, 7	▽ 6/4/79
BORING COMPLETED 6/4/79	19.5		



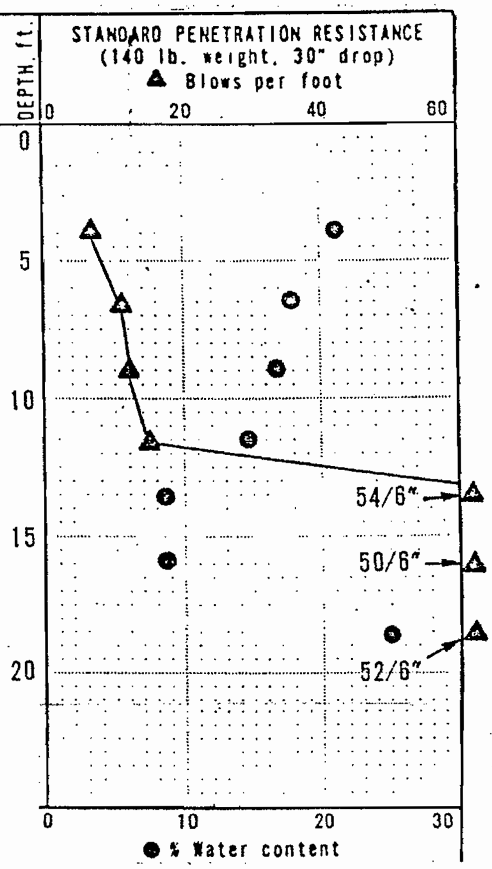
J-28

SOIL DESCRIPTION Surface Elevation: Approx. 309 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Brown, gravelly, sandy SILT.	0		
Stiff, gray, sandy, silty CLAY; trace of gravel.	3	1	
Stiff, brown, sandy, clayey SILT; trace of organics and gravel.	6	2	6/4/79
Medium dense to dense, gravelly, clayey, silty SAND.	7.5	3 4 5 6	
Very dense, gray, gravelly, clayey, silty SAND (GLACIAL TILL)	17	7	
BORING COMPLETED 6/1/79	18.5		



J-29

SOIL DESCRIPTION Surface Elevation: Approx. 307 ft.	DEPTH, ft.	SAMPLES	GROUND WATER
Loose, brown, sandy SILT; scattered organics and gravel (FILL)	0	1	
Medium dense, gray-brown to gray, silty SAND; trace of gravel and clayey lenses. From 7.5 to 10 feet: Stiff, gray-brown, sandy, clayey SILT.	5	2 3 4	6/4/79 (ESTIMATED)
Very dense, gray to gray-brown, gravelly, clayey, silty SAND (GLACIAL TILL)	12.5	5 6	
Hard, mottled gray and brown, silty CLAY; occasional silt and sand partings.	17	7	
BORING COMPLETED 6/4/79	19.0		



APPENDIX E
Site Specific Seismic Hazard Analysis

APPENDIX E
SITE SPECIFIC SEISMIC HAZARD ANALYSIS

Development of the Site-Specific Response Spectra

General

A site specific seismic hazard analysis per ASCE 41-06 Section 1.6.2 was completed to develop the site-specific BSE-1 and BSE-2 response spectra for use in the retrofit design of the Nursing Tower and CLC Building. For this project, fully probabilistic response spectra were developed using the published ground motion prediction equations (GMPEs) and by completing probabilistic 1-dimensional site specific response analysis. The seismic hazard calculation and site specific response analysis were completed by Dr. Walter Silva of Pacific Engineering and Analysis (PE&A) as a subconsultant to GeoEngineers.

Development of fully probabilistic site-specific response spectra typically involves a two-step process. In the first phase or step, the PSHA is conducted for a generic (or reference) site condition that represents the project site defined by the GMPEs used to quantify the ground motions (5 percent damped acceleration response spectra) for a given earthquake magnitude, rupture mechanism and distance to the site. The generic (or reference) site condition may be selected as either rock or soil or defined by the site Vs30 value, with the resulting PSHA producing hazard curves (one for each structural frequency) that quantify the mean annual exceedance frequency (AEF) for a range in spectral accelerations. The hazard curves, as a result of the analysis, properly accommodate the uncertainty and randomness associated with the earthquake sources (for example, magnitude, recurrence, mechanism and depth) and the propagation path from the source to the site, as well as the generic (or reference) site conditions.

For application to the VA Seattle site, the source characterization developed by the USGS (2008 USGS Seismic Hazard Model) was implemented because it reflects a general consensus of recognized experts in the local and regional earthquake source processes. The source characterization accommodated potential contributions from the recognized local and regional earthquakes, including local crustal sources as well as both interplate and intraplate CSZ sources. The generic site condition selected was rock or Vs30 of 600m/sec, consistent with the general classification of the VA Seattle site.

However, the actual site, although considered rock, will differ from the generic rock site conditions represented by the strong motion databases used in developing empirical GMPEs because the project site subsurface soils and rock units may have nonlinear dynamic material properties that differ from the more typical soils and rock conditions in the database. In addition, there is a largely unknown depth to the basement rock material (Vs ~ 2.5 km/s) ranging from about 2,000 feet to about 23,000 feet.

As a result of the differences between the generic rock accommodated in the empirical GMPEs and the VA Seattle site, the generic site PSHA was adjusted for site-specific properties using a fully probabilistic approach. This approach preserves the desired hazard level (AEF) of the generic site PSHA while incorporating both epistemic (uncertainty) and aleatory (randomness) variabilities in site-specific dynamic material properties, as described in Attachment A. The approach makes use of suites of amplification factors (5 percent damped spectral acceleration [Sa]) computed for the specific site, relative to the generic site, for a range in generic site spectral amplitudes (see Attachment A). Separate suites of amplification factors were computed for different sets of nonlinear dynamic material properties to accommodate uncertainty (epistemic variability) in base-case properties or models, as described in Attachment B. Randomness about each base-case model to accommodate random variability, both vertically and across the site, is treated with multiple realizations (typically 30) about each base-case model (for example, shear-wave velocity profile or set of modulus reduction and hysteretic damping curves).

Seismic Hazard Analysis Methodology

For this project, the PSHA was conducted for a generic site of soft rock with an estimated Vs30 of 600m/sec followed by a fully probabilistic adjustment for site-specific conditions and their associated uncertainties. The site specific horizontal and vertical response spectra were then developed using the following procedure:

1. Develop site-specific horizontal response spectra at two foundation levels, one at the ground surface and the other at 15 feet below the ground surface. This was completed by first performing a PSHA using generic rock conditions defined by the published GMPEs. The resulting hazard curves were then adjusted for site-specific subsurface conditions using the results of fully probabilistic site specific response analysis, which preserves the desired exceedance frequencies of the PSHA. To accommodate the large uncertainty in the deeper portion of the local rock, below depths of the site investigations, a 50 percent weight was applied to the generic soil PSHA with 50 percent weight to the site-specific hazard. The calculation was completed for the period range defined by the selected attenuation relations, 0 to 3 seconds.
2. Develop ratios of vertical to horizontal response spectra using the GMPE by Abrahamson & Silva (1997) and Campbell & Bozorgnia (2003) for period range of 0 to 3 seconds. The vertical response spectra were computed by integrating the V/H ratios with the horizontal response spectra.

The following sections present a summary of each of the procedures outlined above.

Development of Site Specific Horizontal Response Spectra

GMPE APPROACH

Using the 2008 USGS Seismic Hazard Source Model, the earthquake hazard at the project site was calculated using a suite of selected GMPEs. The earthquake hazard calculation was completed by Walt Silva of PE&A using the computer program HAZ-38. This computer program has been validated in the Pacific Earthquake Engineering Research (PEER)-sponsored “Validation of PSHA Computer Programs” Project (Wong et al., 2004) and qualified for use by the U.S. Department of Energy.

The 2008 USGS Seismic Hazard Model contains seismic source geometries and recurrence models developed by USGS in the 2008 Update of the United States National Seismic Hazard Maps. For details on the 2008 USGS Seismic Hazard Model, please refer to USGS Open-File Report 2008-1128 (Petersen et al., 2008).

The GMPEs used in the seismic hazard calculation were selected based on the project site conditions and the tectonic environment for which the equations were developed. The following table summarizes the GMPEs selected for use in the hazard calculation for this project and the weighting assigned to each equation.

Earthquake Sources	Ground Motion Prediction Equations	Site Conditions	Weight
Crustal	Abrahamson & Silva NGA (2008)	Vs30 = 600 m/sec	0.25
	Boore & Atkinson NGA (2008)	Vs30 = 600 m/sec	0.25
	Campbell & Bozorgnia NGA (2008)	Vs30 = 600 m/sec	0.25
	Chiou & Youngs NGA (2008)	Vs30 = 600 m/sec	0.25
Benioff / Intraplate	Atkinson & Boore (2003)	Rock	0.34
	Youngs et al. (1997)	Rock	0.33
	Zhao et al. (2006)	Rock	0.33
CSZ	Atkinson & Boore (2003)	Rock	0.30
	Youngs et al. (1997)	Rock	0.30
	Zhao et al. (2006)	Rock	0.30
	Gregor et al. (2002)	Rock	0.10

Calculation of the earthquake hazard was completed for a generic rock site using the selected GMPEs. Unlike the GMPEs developed for the Benioff and CSZ earthquakes where generally two site conditions (soil or rock) are considered, the site conditions considered in the NGA relationships are classified using Vs30. Based on the shear wave velocity measurement at the site, a Vs30 of about 600m/sec was selected for the NGA GMPEs. The mean hazard curve for the 475-year (BSE-1) and 2,475-year (BSE-2) return period are constructed and presented in Figure E-1 for the generic site.

SITE RESPONSE ANALYSIS APPROACH

To accommodate the effects of site-specific dynamic material properties on the generic site hazard, a series of site response analyses were performed. The site response analyses produce suites of amplification factors (5 percent damped response spectra) for the VA Seattle site relative to the generic site. To develop the amplification factors, the conventional equivalent-linear approximation to nonlinear soil response was used along with vertically propagating shear-waves. In the site response approach implemented here, time histories were not used. Instead, a frequency domain random vibration theory (RVT) approach was implemented that requires only the control motion power spectrum. The methodology is discussed in Appendix B. Control motions used to drive the soil columns were generated with the point-source model (EPRI, 1993; Silva et al., 1997, Walling et al., 2008). The site-specific UHS was developed from the generic site hazard using an approach that correctly preserves the probability or hazard level of the reference site PSHA (Bazzurro and Cornell, 2004; Approach 3, Attachment A). This approach produces an accurate site-specific mean hazard that accommodates both aleatory (randomness) as well as epistemic (uncertainty) variabilities in dynamic material properties across the site.

Aleatory variability (randomness in dynamic material properties such modulus reduction and hysteretic damping curves as well as shear-wave velocity profile) resulting from both lateral and vertical random variations about base-case values is accommodated by randomly varying properties to generate distributions for the amplification factors. The distributions of the amplification factors are then integrated with the generic site hazard curves to produce the site-specific hazard curves (Attachment A). The profile randomization scheme uses a model based on an analysis of variance of about 500 measured shear-wave velocity profiles (EPRI, 1993, Attachment B). As with the shear-wave velocity profiles, the G/Gmax and hysteretic damping curves are randomized about base-case values to accommodate aleatory variability vertically as well as across the site (Attachment B).

Epistemic variability (uncertainty in mean or base-case models) in dynamic material properties as well as potential basin effects are accommodated by developing separate suites of amplification factors for each model along with their corresponding randomness (aleatory variability) about each base-case model.

Finally, the site-specific hazard is computed by integrating the distributions (median and sigma estimates) of the amplification factors with the generic soil site hazard curves. This process correctly accommodates the site-specific randomness (aleatory variability) of dynamic material properties across the site in the development of site-specific (soil) hazard curves. For each suite of base-case properties (epistemic variability), site-specific mean hazard curves were developed that properly include randomness (aleatory variability) about the base-case properties resulting from the 30 realizations of random dynamic material properties. The resultant hazard curves, one for each base-case, are then averaged over exceedance frequency, resulting in a single site-specific hazard curve at each structural frequency. In the averaging process, weights are employed reflecting the likelihood of in-situ conditions for modulus reduction and hysteretic damping curves as well as potential basin effects.

BASE-CASE VS PROFILES AND DYNAMIC SOIL CURVES

In order to compute the amplification factors, base-case profiles and dynamic soil curves (G/G_{max} and hysteretic damping) were developed for both the generic site conditions and project site condition. For the generic site condition, the Vs_{30} of 600m/sec shear-wave velocity profile incorporated in developing the analytical NGA amplification factors (Walling et al., 2008) was used in computing the generic site amplification factors.

To develop site-specific shear-wave velocity profiles, the measured shear-wave velocity profile at the project site and from the Seward Park site located near the project site were used. The shear-wave velocity data for the project site was obtained to depth of about 60 feet where practical refusal was encountered during drilling. The shear wave velocity data for the Seward park site was obtained to depth of about 100 feet. The shear wave velocity data obtained at the project site and the Seward Park site were incorporated into the development of the site-specific shear-wave velocity for the near surface soil for the VA Hospital site.

The local and regional profiles were used to develop a range in possible shear-wave velocities at the site at depth greater than 100 feet. The range in velocities is represented by three profiles reflecting a best estimate as well as upper- and lower-range profiles as shown in Figures E-2 and E-3. These three profiles were developed by reviewing the sonic velocities measured in the oil test well logs completed in the Puget Sound and by reviewing the Seattle basin model developed by Frankel et al. (2007) and Pratt et al. (2003). The three profiles presented in Figures E-2 and E-3 were developed to accommodate the uncertainties related to the 1-D amplification corresponding to Seattle basin.

For the nonlinear dynamic material properties, generic soil G/G_{max} and hysteretic damping curves were used (EPRI, 1993). Based on the Abrahamson and Silva (2008) regression analyses on recorded motions, the more linear Peninsular Range curves (Silva et al., 1997) performed slightly better than the more nonlinear EPRI (1993) set of curves and were used to develop the amplification factors. The Peninsular Range curves use the EPRI (1993) 51- to 120-foot curves for depths of 0 to 50 feet and the 501- to 1,000-foot curves for deeper materials. Both the shear-wave velocity profile and the soil modulus reduction and hysteretic damping curves were randomized (Attachment B) to account for the aleatory variability over the site area.

AMPLIFICATION FACTORS

The site-specific amplification, relative to the generic site, was characterized by a suite of frequency-dependent amplification factors that account for nonlinearity in soil response. Amplification factors were computed by propagating magnitude (**M**) 5.0, 6.0 and 7.0 control motion power spectra using the single-corner point-source model through each combination of the randomized shear-wave velocity profiles and material curves. To adequately cover the ground motion ranges in the generic soil site hazard curves, amplification factors were computed for a suite of 11 expected generic site peak acceleration values (0.01g, 0.05g, 0.10g, 0.20g, 0.30g, 0.40g, 0.50g, 0.75g, 1.00g, 1.25g, 1.50g). These amplification factors were computed at the same periods as the generic site spectral acceleration (3.0 sec, 2.0 sec, 1.5 sec, 1.0 sec, 0.75 sec, 0.50 sec, 0.30 sec, 0.20 sec, 0.10 sec, 0.075 sec, 0.01 sec (PGA)). For each suite of base-case properties (epistemic variability) (for example, profile (6), curves (2) and magnitude (3)), site-specific mean hazard curves were developed that properly include randomness (aleatory variability) about the base-case properties. The resultant hazard curves, one for each base-case, were then averaged over exceedance frequency, resulting in a single site-specific mean hazard curve at each structural frequency. In the averaging process, weights were employed reflecting the likelihood of in-situ conditions for each profile as well as set of modulus reduction and hysteretic damping curves. Site-specific hazard curves developed from amplification factors computed for each magnitude (**M** 5, 6, 7) were weighted by the model deaggregations for each structural frequency at AEF 2×10^{-3} (475-year return period) and 4×10^{-4} (2,475-year return period). In accommodating all the base-case models, multiple magnitudes, 11 ground motion levels, and randomness through multiple realizations required over 10,000 site response analyses.

SITE SOIL RESPONSE SPECTRA

Site-specific mean UHS (5 percent damped) at AEF 2×10^{-3} (475-year return period) and 4×10^{-4} (2,475-year return period) were developed by integrating the suites of amplification factors with the generic site hazard curves and are shown in Figure E-4.

Site-Specific Horizontal Response Spectra

As shown in Figures E-1 and E-4, the site-specific UHS reflects the generally stiff project site condition relative to the generic site profile, showing increased short period ($T \leq 0.3$ seconds) and decreased long period motions relative to the generic site UHS. Given the degree of uncertainty regarding the soil profiles at the site and within the vicinity, equal weight was given to the site-specific and generic soil hazard in developing the VA Seattle site-specific UHS. The site-specific horizontal response spectra based on this approach is presented in Figure E-5 at the ground surface and at depth of 15 feet.

Development of Site Specific Vertical Response Spectra

- The most commonly used approach in developing the vertical response spectra is to apply a constant ratio of vertical to horizontal (V/H) response spectra of 0.65. Based on the studies completed by Campbell & Bozorgnia (2003) and others, this factor is found to be not conservative for site located near the causative fault, such as the VA Hospital site. For this project, site specific V/H ratios were computed and incorporated in developing the site specific vertical response spectra.
- The V/H ratio varies as a function of structural period, distance to the fault, earthquake magnitude and site conditions. Based on the results of the seismic hazard deaggregation of the site, the seismic hazard for the period range of 0 to 3 seconds is strongly influenced by Magnitude 7 earthquakes with distance to the fault of 1 km and 50 km. Two GMPEs, one by Abrahamson & Silva (1997) and the other by Campbell & Bozornia (2003), were used to calculate the V/H ratio for period range of 0 to 3 seconds. A generic rock site condition was assumed in the calculation. The results of V/H ratio calculated using the two GMPEs are presented in Figure E-6.
- Site specific V/H ratios were developed by integrating the results shown in Figure E-6 probabilistically. The results calculated by the two GMPEs were weighted equally. The relative weights for each of the scenario considered were determined with the results of the seismic hazard deaggregation. The relative weights for the 1 km and 50 km distance is 0.55 and 0.45, respectively for the BSE-1 (475-year return period) event. For the BSE-2 (2,475-year return period) event, the relative weights for the 1 km and 50 km distance is 0.80 and 0.20, respectively.
- The site-specific vertical response spectra were then computed by applying the site specific V/H ratios to the site specific horizontal response spectra as presented in Figure E-5. The site specific vertical response spectra at the ground surface and at depth of 15 feet are presented in Figure E-7.

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Appendix E-A
Approaches to Develop Site Specific Hazard

APPENDIX E-A
APPROACHES TO DEVELOP SITE-SPECIFIC HAZARD

In developing site-specific UHRSs or hazard there are two goals that must be met to achieve desired risk levels:

1. Preserve the hazard level (AEF) of the reference site PSHA across structural frequency (hazard consistent).
2. Incorporate site-specific aleatory (randomness) and epistemic (uncertainty) variabilities of dynamic material properties in the hazard.

Description of Approaches

In general, there are four fairly distinct approaches intended to accomplish the stated goals. The approaches range from the simplest and least accurate, which scales the reference site UHRS on the basis of a site-response analysis using a broad-band control motion (Approach 1), to the most complex and most accurate, a PSHA computed using attenuation relations, median estimates and standard deviations, developed for the specific site (Approach 4).

Approach 1

This approach is fundamentally deterministic and involves, for a rock references site, use of the outcrop UHS to drive the site-specific column(s). It assumes that a rock outcrop hazard (UHS) has similar characteristics as rock beneath soil, which is not generally a valid assumption for soft rock (NUREG/CR-6728), and has no mechanism to conserve the outcrop AEF. For cases where the hazard is dominated by earthquakes with significantly different magnitude (M) at low (for example, ≤ 1 Hz to 2.5 Hz) and high (e.g. ≥ 5 Hz to 10 Hz) structural frequencies, the outcrop UHS may be quite broad, unlike any single earthquake, resulting in unconservative high-frequency motions (too nonlinear in site response). Even if only a single earthquake is the major contributor at all structural frequencies, variabilities incorporated in the hazard analysis may result in a broad spectrum, again unlike any single earthquake. For these reasons, this approach is discouraged, and an alternative semi-deterministic method (such as Approach 2) may be used.

Approach 2

This approach is also fundamentally deterministic and is intended to avoid the broad-band control motion of Approach 1. For a rock reference site, Approach 2 uses low- and high-frequency (and intermediate if necessary) deterministic spectra computed from the attenuation relations used in the PSHA, or suitable spectral shapes (NUREG/CR-6728), reflecting expected rock conditions beneath the local soils, scaled to the UHRS at the appropriate frequencies (for example, RG 1.165). These scaled motions, computed for the modal deaggregation M and D are then used as control motions to develop multiple (typically 2 to 3) mean transfer functions based on randomized soil columns. If the control motions are developed from the attenuation relations used in the reference PSHA, the generic site condition they reflect must be appropriate for the rock beneath the local soils. Additionally, separate control motions should be developed for each attenuation relation to include the effects of spectral shape uncertainty (epistemic) on soil response. The resulting mean transfer functions would then be combined using the same relative weights as in the reference PSHA. The mean transfer functions are then enveloped with the resulting transfer function applied to the outcrop (rock or soil) UHS.

This method was termed Approach 2A in NUREG/CR-6728. The use of mean (rather than median) transfer functions followed by enveloping is an empirical procedure to conservatively maintain the outcrop exceedance probability (NUREG/CR-6728 and –6769), because this fundamentally deterministic approach does not include the contributions to soil spectra from the entire range in rock or reference site hazard (Bazzurro and Cornell, 2004). The motivation for this "empirical" procedure is discussed in Approach 3 – Approximate Method.

For cases where there may be a wide magnitude range contributing to the hazard at low- or high-frequency and/or the site has highly nonlinear dynamic material properties, low, medium, and high M control motion spectra may be developed at each frequency of interest. A weighted mean transfer function (for example, with weight of 0.2, 0.6, 0.2 reflecting 5 percent, mean, 95 percent M contributions) is then developed at each structural frequency of interest. Following Approach 2A, the weighted mean transfer functions for each frequency of interest are then enveloped with the resultant value applied to the outcrop UHS. This more detailed analysis procedure was termed Approach 2B.

Approach 3

This approach is a fully probabilistic analysis procedure which moves the site response, in an approximate way, into the hazard integral. The approach is described by Bazzurro and Cornell (2004) and NUREG/CR-6769. In this approach, the hazard at the soil surface is computed by integrating the site-specific hazard curve at the bedrock level with the probability distribution of the amplification factors (Lee et al., 1998; 1999). The site-specific amplification, relative to CENA rock is characterized by a suite of frequency-dependent amplification factors that can account for nonlinearity in soil response. Approach 3 involves approximations to the hazard integration using suites of transfer functions, which result in complete hazard curves at the ground surface, or any other location, for specific ground motion parameters (for example, spectral accelerations) and a range of frequencies.

The basis for Approach 3 is a modification of the standard PSHA integration:

$$P[A_S>z] = \int\int\int P\left[AF > \frac{z}{a} \middle| m,r,a\right] f_{M,R|A}(m,r;a) f_A(a)\,dm\,dr\,da \qquad (1)$$

where AS is the random ground motion amplitude on soil at a certain natural frequency, z is a specific level of AS, m is earthquake magnitude, r is distance, a is an amplitude level of the random reference site (for example, hard rock) ground motion, A, at the same frequency as AS, fA(a) is derived from the rock hazard curve for this frequency (namely, it is the absolute value of its derivative), and fM,R|A is the deaggregated hazard (that is, the joint distribution of M and R, given that the rock amplitude is level a). AF is an amplification factor defined as:

$$AF = A_S/a \qquad (2)$$

where AF is a random variable with a distribution that can be a function of m, r and a. To accommodate epistemic uncertainties in site dynamic material properties, multiple suites of AF may be used and the resulting hazard curves combined with weights to properly reflect mean hazard and fractiles.

Soil response, in terms of site amplification (Sa (site)/Sa (reference)), is controlled primarily by the level of rock motion and m, so Equation 1 can be approximated by:

$$P[A_S>z] = \int\int P[AF > \frac{z}{a}(m,a) f_{M|A}(m;a) f_A(a) dmda \qquad (3)$$

where r is dropped because it has an insignificant effect in most applications. To implement Equation 3, only the conditional magnitude distribution for relevant amplitudes of a is needed. fM|A(m;a) can be represented (with successively less accuracy) by a continuous function, with three discrete values or with a single point (for example, m1(a), the model magnitude given a). With the latter, Equation 3 can be simplified to:

$$P[A>z] = \int\int P[AF > \frac{z}{a} | a, m^1(a)] f_A(a) da \qquad (4)$$

where, fM|A(m;a) has been replaced with m1 derived from deaggregation. With this equation, one can integrate over the rock acceleration, a, to calculate P[AS>z] for a range of soil amplitudes, z.

It is important to note there are two ways to implement Approach 3. The full integration method described below or simply modifying the attenuation relation ground motion value during the hazard analysis with a suite of transfer functions (Cramer, 2003). Both implementations result in very similar site-specific hazard (Cramer, 2003), and both will tend to double-count site aleatory variability, once in the suite of transfer function realizations and again in the aleatory variability about each median attenuation relation. The full integration method tends to lessen any potential impacts of the large total site aleatory variability (Bazzuro and Cornell, 2004). Approximate corrections, for the site component of aleatory variability, may be made by implementing the approximate technique (Equation 7) with C = 0, AF =1, and a negative exponential, where arp = the soil amplitude and σ the component of variability that is removed. For the typical aleatory variability of the amplification factors ($\sigma ln \approx 0.1$-0.3) and typical hazard curve slopes in the CENA ($\kappa \approx 2$-3), the reduction in motion is about 5 percent to 10 percent.

Approach 4

Approach 4 entails the development and use of site-specific attenuation relationships, median estimates and aleatory variabilities, developed specifically for the site of interest, which incorporate the site response characteristics of the site. The PSHA is performed using these site-specific relationships for the specified AEF. This approach is considered the most accurate because it is intended to accommodate the appropriate amounts of aleatory variability into site- and region-specific attenuation relations. Epistemic variability is appropriately captured through the use of multiple attenuation relations. Approach 3 is considered as a fully probabilistic approximation to Approach 4.

Approach 3 – Full Integration Method

The site-specific hazard curve can be calculated using the discretized form of Equation 3 from Bazzurro and Cornell (2004).

$$G_Z(z) = \sum_{all\ x_j} P\left[Y \geq \frac{z}{x}\middle| x_j\right] p_X(x_j) = \sum_{all\ x_j} G_{Y|X}\left(\frac{z}{x}\middle| x_j\right) p_X(x_j). \tag{5}$$

where $G_Z(z)$ is the sought hazard curve for Ssa(f), that is, the annual probability of exceeding level z.

$$G_{Y|X}\left(\frac{z}{x}\middle| x\right) = \hat{\Phi}\left(\frac{ln\left[\frac{z}{x}\right] - ln\left[\hat{m}_{Y|X}(x)\right]}{\sigma_{\ln Y|X}}\right) \tag{6}$$

where $G_{Y|X}$ is the complementary cumulative distribution function of (CCDF) Y = AF(f), conditional on a rock amplitude x. This is simply the CCDF of the site amplification factors as a function of control motion (for example, rock or reference site) loading level.

$\hat{\Phi}$ = 1 - Φ - the widely tabulated complementary standard Gaussian cumulative distribution function.

$\hat{m}_{Y|X}$ - the conditional median of Y (the amplification factor).

$\sigma_{\ln Y|X}$ - the conditional standard deviation of the natural logarithm of Y (aleatory variability of the amplification factor).

$p_X(x_j)$ - the probability that the rock or reference site control motion level is equal to (or better, in the neighborhood of) x_j.

Equation 5 is the essence of Approach 3 and simply states that the soil hazard curve is computed as the product of the soil amplification (specifically, its CCDF), conditional on a reference (rock) amplitude x, times the probability of obtaining that reference amplitude, summed over all reference amplitudes.

The soil amplifications, median and σln estimates are all that is required and are generated by driving the soil column at a suite of reference site motions. At each reference motion, multiple realizations of randomized dynamic material properties are developed followed by site response analyses to generate a suite, typically 30 to 100, of amplification factors. From that suite, a median and σln are computed, generally assuming a log-normal distribution.

The probability of obtaining a reference motion is simply the derivative of the reference (for example, rock) hazard curve obtained from the PSHA. This is done numerically and is a stable process as the hazard curves are quite smooth. Equation 5 can quite easily be put into an Excel spreadsheet. It forms the entire basis of our FORTRAN code. Approach 3 is indeed one simple equation. This approach is implemented in the computer program SOILUHSI.

Approach 3 – Approximate Method

An alternative solution to Equation 4 can also be calculated using Equation (7) from Bazzuro and Cornell (2004). This is a closed form approximation of the integration of the amplification factor over a range of rock amplitudes.

$$z_{rp} = a_{rp} \overline{AF_{rp}} \exp \left(\frac{\boldsymbol{\sigma}_{\boldsymbol{\delta}}^{2}}{\mathbf{2}} \frac{\kappa}{\mathbf{1-C}}\right) \qquad (7)$$

where zrp is soil amplitude z associated with return period rp; arp is the reference spectral acceleration a associated with return period rp; $\overline{\text{AFrp}}$ is the geometric mean (mean log) amplification factor for the reference (for example, rock) motions with return period rp; k is the log-log slope of the reference hazard curve that is calculated at each point from the reference hazard curve and typically ranges from about 2 to 3 for CENA and possibly as large as 6 for WNA. C is the log-log slope (absolute value) of the amplification factor with respect to the reference motion that is calculated at each point from the amplification factors (AF) and is a measure of the degree of soil nonlinearity. If C = 0, the response is linear and highly nonlinear for C approaching 1, where the approximation breaks down (Bazzurro and Cornell, 2004). As previously mentioned, C typically ranges from about 0.1 to about 0.8 (Bazzurro and Cornell, 2004). σ_{δ} is the log standard deviation of the AF and is typically around 0.3 (σln) or less. In other words, at a given AEF or point on the reference site hazard curve, the corresponding soil amplitude is given as the median soil amplification times the rock or reference site amplitude plus an exponential factor.

The exponential factor is necessary to maintain the reference AEF and accommodates both the aleatory variability as well as the degree of nonlinearity of the site amplification. The slope of the reference hazard curve is a weighting factor that includes the contributions to the soil amplitude for all reference hazard levels. Equation 7 clearly demonstrates the additional factors needed over median amplification to preserve the hazard level (AEF) of the reference motion. This Equation shows that in order to preserve the reference site (for example, rock) hazard level, multiplying the reference motion by the median soil amplification requires an additional exponential term. This additional term includes the aleatory variability of the soil or amplification factor, the slope of the reference site hazard curve, and the slope of the amplification factors (for example, with varying reference motion). This exponential factor accommodates the potential contributions to a given soil motion by the entire range in reference site motions because of soil nonlinearity. That is, a given soil motion may have the same value at low levels of reference loading (relatively linear response) and at high loading levels (relatively nonlinear response).

To preserve the reference site exceedance frequency, all the contributions to a given soil motions over the entire range in reference loading levels must be included in the soil hazard. These contributions are not explicitly considered in the deterministic Approach 2 method. Additionally, the effects of aleatory variability in the soil amplification resulting from lateral variability in velocities and depth to basement as well as randomness in G/Gmax and hysteretic damping curves are included in the exponential term. For a linear site, C is zero, so it is easy to see that the exponential term then accommodates the effects of profile variability in the soil hazard. The reference hazard curve slope (κ in Equation 7) is present to accommodate the impacts of the soil variability and nonlinear amplification over the entire reference site motion or hazard curve.

In the case C = 0 and for a reference hazard slope near 1, the median amplification times the exponential term simply reflects the mean, for a lognormal distribution.

This was the motivation for using mean, rather than median amplification factors in Approach 2. However, for more realistic reference site hazard curve slopes, use of the mean amplification alone will result in motions that are too low for the assumed AEF. The difference or underestimate increases as soil nonlinearity, characterized through C, becomes larger for a given aleatory variability in the amplification factors. This was the motivation for the "empirical" correction in Approach 2 of enveloping the low- and high-frequency transfer functions. The high-frequency transfer function will typically have lower high-frequency amplification than the low-frequency amplification factor as it reflects higher loading levels, resulting in a higher degree of nonlinearity, and a greater value of C. Use of mean amplification alone may then depart significantly from Equation 7, resulting in higher probability motions than would be consistent with the reference hazard level, depending on the value of C and the slope of the reference hazard curve. Using an envelop of the low-frequency amplification, which typically does not reflect nearly as high loading levels at high frequency, and the high-frequency amplification was an ad-hoc manner of conservatively achieving the desired AEF using deterministic analyses.

It is important to point out that a similar issue, though less significant, can occur at low frequency. In this case, the high-frequency amplification has larger low-frequency amplification than the low-frequency amplification. The envelope at low frequency is then controlled by the high-frequency amplification, compensating for the neglect of the complete exponential in the low-frequency mean amplification (NUREG/CR-6728). This approach is implemented in the computer program SOILUHS.

Implementation of Approach 3

Approach 3 is implemented using the full integration method which consists simply of coding Equation 5. The soil (or rock) amplification distributions relative to the reference site condition are developed by driving the site-specific column at a suite of distances generated on a grid of expected reference site peak accelerations, to accommodate nonlinear soil response. At each distance, or reference site expected peak acceleration, random suites of dynamic material properties are generated, resulting in a distribution of structural frequency dependent amplification factors (Sa (site)/Sa (reference)). For a given structural frequency, say 1 Hz, this process results in median and sigma estimates, for each loading level, from which a CCDF is produced using standard asymptotic expressions, accurate typically to the fourth decimal place. For each loading level, reference Sa at 1 Hz, the amplification CCDF is then available to integrate over the entire reference 1 Hz hazard curve. This is precisely the motivation for the wide range in reference peak accelerations, 0.01g to 1.50g, to cover the entire reference hazard curve for each structural frequency. For reference site motion outside the range, the closest values are used. To minimize any error in interpolation (log) for reference site motions between grid points, a dense sampling of typically 11 values (for example, 0.01, 0.05, 0.10, 0.20, 0.30, 0.40, 0.50, 0.75, 1.00, 1.25, 1.50g) of expected (median) reference site peak accelerations are used. The array of peak accelerations is sampled more densely over the range in values contributing most to the hazard, typically 0.2g to 0.5g. Because the amplification factors are smooth (Bazzurro and Cornell, 2004; Silva et al., 1999), interpolation is not a significant issue and an 11-point grid is adequate to capture site nonlinearity.

To compute the probability of reference motions (P(x) in Equation 5), the reference motion hazard curve is numerically differentiated using central differences. Although hazard curves are smooth and therefore differencing is a stable process, the curves are interpolated to 300 points to maximize the integration accuracy of Equation 5. The use of 300 points was established by increasing the number of points until stability (no change in derived soil hazard) was achieved. This typically occurred between 100 to 200 points, and so 300 points has been adopted as a conservative value for integration.

Because multiple levels of reference motions contribute to the soil or site-specific hazard, a wider range in reference hazard than soil hazard is necessary to achieve accuracy in the soil hazard. Extensive tests have shown that a conservative range over which to integrate the reference hazard is a factor of 10 in AEF beyond that desired for the soil or site-specific AEF. In other words, if the site-specific hazard is desired to 10-6 AEF, the reference hazard is required to an AEF of 10-7. Additionally, the same consideration applies at high exceedance frequencies as well. In this case, if the site-specific hazard is desired at 10-2 AEF, the reference hazard is conservatively required to an AEF of 10-1.

Approach 3 is also appropriate for computing site-specific vertical hazards from horizontal site-specific hazard curves, producing a vertical UHRS at the same AEF as the horizontal UHRS. Resulting horizontal and vertical UHRSs then both achieve the same target performance goals. As with the horizontal site-specific hazard, regarding the range in the reference site hazard, accuracy in the vertical hazard requires a wide integration range over the site-specific horizontal hazard. As a result, achieving an AEF of 10-6 for the vertical site-specific hazard requires the reference site hazard to an AEF of 10-8.

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Appendix E-B
Site Response Analysis Method

APPENDIX E-B
SITE RESPONSE ANALYSIS METHOD

Development of Site-Specific Soil Motions

The conventional approach to estimating the effects of site-specific site conditions on strong ground motions involves development of a set (1, 2, or 3 component) of time histories compatible with the specified outcrop response spectra to serve as control (or input) motions. The control motions are then used to drive a nonlinear computational formulation to transmit the motions through the profile. Simplified analyses generally assume vertically propagating shear-waves for horizontal components and vertically propagating compression-waves for vertical motions. These are termed one-dimensional site response analyses.

Equivalent-Linear Computational Scheme

The computational scheme that has been most widely employed to evaluate one-dimensional site response assumes vertically-propagating plane shear-waves. Departures of soil response from a linear constitutive relation are treated in an approximate manner through the use of the equivalent-linear approach.

The equivalent-linear approach, in its present form, was introduced by Seed and Idriss (1970). This scheme is a particular application of the general equivalent-linear theory developed by Iwan (1967). Basically, the approach is to approximate a second order nonlinear equation, over a limited range of its variables, by a linear equation. Formally, this is done in such a way that the average of the difference between the two systems is minimized. This was done in an ad-hoc manner for ground response modeling by defining an effective strain that is assumed to exist for the duration of the excitation. This value is usually taken as 65 percent of the peak time-domain strain calculated at the midpoint of each layer, using a linear analysis. Modulus reduction and hysteretic damping curves are then used to define new parameters for each layer based on the effective strain computations. The linear response calculation is repeated, new effective strains evaluated, and iterations performed until the changes in parameters are below some tolerance level. Generally, a few iterations are sufficient to achieve a strain-compatible linear solution.

This stepwise analysis procedure was formalized into a one-dimensional, vertically propagating shear-wave code called SHAKE (Schnabel et al., 1972). Subsequently, this code has easily become the most widely used analysis package for one-dimensional site response calculations.

The advantages of the equivalent-linear approach are that parameterization of complex nonlinear soil models is avoided and the mathematical simplicity of a linear analysis is preserved. A truly nonlinear approach requires the specification of the shapes of hysteresis curves and their cyclic dependencies through an increased number of material parameters. In the equivalent-linear methodology, the soil data are utilized directly and, because at each iteration the problem is linear and the material properties are frequency independent, the damping is rate independent and hysteresis loops close.

Careful validation exercises between equivalent-linear and fully nonlinear formulations using recorded motions from 0.05 to 0.50g showed little difference in results (EPRI, 1993). Both formulations compared very favorably to recorded motions, suggesting both the adequacy of the vertically propagating shear-wave model and the approximate equivalent-linear formulation. Although the assumptions of vertically propagating shear-waves and equivalent-linear soil response certainly represent approximations to actual conditions, their combination has achieved demonstrated success in modeling observations of site effects and represent a stable, mature and reliable means of estimating the effects of site conditions on strong ground motions (Schnabel et al., 1972; Silva et al., 1988; Schneider et al., 1993; EPRI, 1993).

To accommodate both uncertainty and randomness in dynamic material properties, analyses are typically done for the best estimate shear-wave velocity profile as well as upper- and lower-range profiles. The upper and lower ranges are usually specified as twice and one-half the best estimate shear-wave moduli. Depending upon the nature of the structure, the final design spectrum is then based upon an envelope or average of the three spectra.

For vertical motions, the SHAKE code is also used with compression-wave velocities and damping substituted for the shear-wave values. To accommodate possible nonlinear response on the vertical component, because modulus reduction and hysteretic damping curves are not generally available for the constrained modulus, the low-strain Poisson's ratio is usually fixed and strain compatible compression-wave velocities calculated using the strain compatible shear moduli from the horizontal component analyses combined with the low-strain Poisson's ratios. In a similar manner, strain compatible compression-wave damping values are estimated by combining the strain compatible shear-wave damping values with the low-strain damping in bulk or pure volume change. This process assumes that the loss in bulk (volume change) is constant or strain independent. Alternatively, zero loss in bulk is assumed and the following equation relating shear- and compression-wave damping (ηS and ηP) and velocities (VS and VP) is used:

$$\eta_P \approx \frac{4}{3} \frac{V_S}{V_P} \eta_S \, , \qquad \text{(B-1)}$$

RVT-Based Computational Scheme

The computational scheme employed to compute the site response for this project uses an alternative approach employing random vibration theory (RVT). In this approach, the control motion power spectrum is propagated through the one-dimensional soil profile using the plane-wave propagators of Silva (1976). In this formulation, only SH waves are considered. Arbitrary angles of incidence may be specified, but normal incidence is used throughout the present analyses.

In order to treat possible material nonlinearities, an RVT-based equivalent-linear formulation is employed. Random process theory is used to predict peak time domain values of shear-strain based upon the shear-strain power spectrum. In this sense, the procedure is analogous to the program SHAKE except that peak shear-strains in SHAKE are measured in the time domain. The purely frequency domain approach obviates a time domain control motion and, perhaps just as significant, eliminates the need for a suite of analyses based on different input motions.

This arises because each time domain analysis may be viewed as one realization of a random process. Different control motion time histories reflecting different time domain characteristics but with nearly identical response spectra can result in different nonlinear and equivalent-linear response.

In this case, several realizations of the random process must be sampled to have a statistically stable estimate of site response. The realizations are usually performed by employing different control motions with approximately the same level of peak accelerations and response spectra.

In the case of the frequency domain approach, the estimates of peak shear-strain as well as oscillator response are fundamentally probabilistic in nature, as a result of the random process theory. For fixed material properties, stable estimates of site response can then be obtained with a single run.

In the context of the RVT equivalent-linear approach, a more robust method of incorporating uncertainty and randomness of dynamic material properties into the computed response has been developed. Because analyses with multiple time histories are not required, parametric variability can be accurately assessed through a Monte Carlo approach by randomly varying dynamic material properties. This results in median as well as other fractile levels (for example, 16th, mean, 84th) of smooth response spectra at the surface of the site. The availability of fractile levels reflecting randomness and uncertainty in dynamic material properties then permits a more rational basis for selecting levels of risk.

In order to randomly vary the shear-wave velocity profile, a profile randomization scheme has been developed that varies both layer velocity and thickness. The randomization is based on a correlation model developed from an analysis of variance on about 500 measured shear-wave velocity profiles (EPRI, 1993; Silva et al., 1997). Profile depth (depth to competent material) is also varied on a site-specific basis using a uniform distribution. The depth range is generally selected to reflect expected variability over the structural foundation as well as uncertainty in the estimation of depth to competent material.

To model parametric variability for compression-waves, the base-case Poisson's ratio is generally fixed. Suites of compatible random compression- and shear-wave velocities are then generated based on the random shear-wave velocities profiles.

To accommodate variability in modulus reduction and hysteretic damping curves on a generic basis, the curves are independently randomized about the base case values. A log normal distribution is assumed with a σ_{ln} of 0.35 at a cyclic shear strain of 3 x 10-2 percent. These values are based on an analysis of variance on a suite of laboratory test results. An upper and lower bound truncation of 2σ is used to prevent modulus reduction or damping models that are not physically possible. The random curves are generated by sampling the transformed normal distribution with a σ_{ln} of 0.35, computing the change in normalized modulus reduction or percent damping at 3 x 10-2 percent shear strain, and applying this factor at all strains. The random perturbation factor is reduced or tapered near the ends of the strain range to preserve the general shape of the median curves (Silva, 1992).

To model vertical motions, incident inclined compression- and shear (SV)-waves are assumed. Raytracing is done from the source location to the site to obtain appropriate angles of incidence. In the P-SV site response analyses, linear response is assumed in both compression and shear with the low-strain shear-wave damping used for the compression-wave damping (Johnson and Silva, 1981). The vertical and horizontal motions are treated independently in separate analyses. Validation exercises with a fully three-dimensional soil model using recorded motions up to 0.50%g showed these approximations to be validated (EPRI, 1993).

In addition, the site response model for the vertical motions has been validated at over 100 rock and soil sites for three large earthquakes: the 1989 M 6.9 Loma Prieta, 1992 M 7.2 Landers, and 1994 Northridge earthquakes. In general, the model performs well and captures the site and distance dependency of vertical motions over the frequency range of about 0.3 to 50.0 Hz and the fault distance range of about 1 to 100 km.

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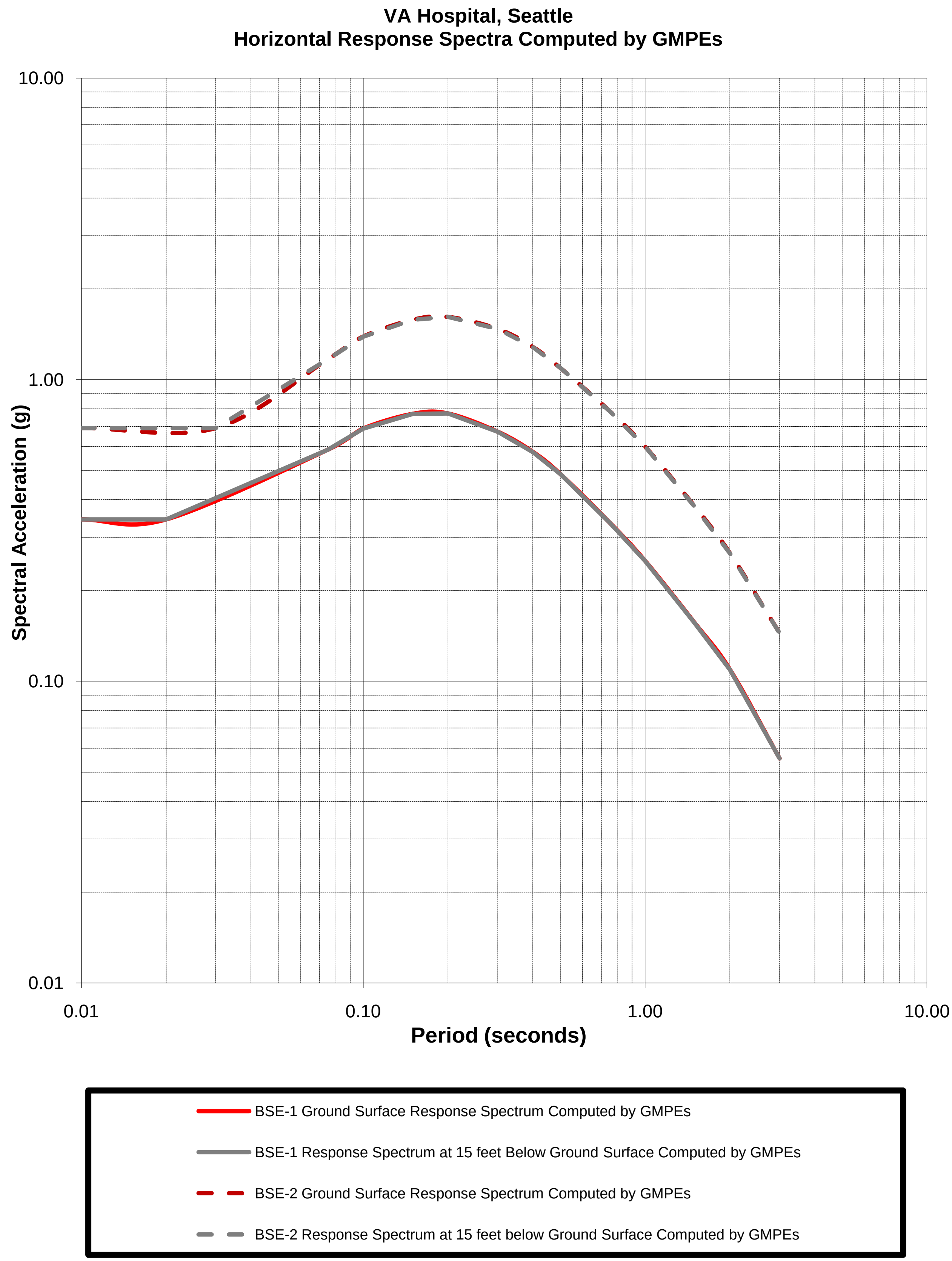

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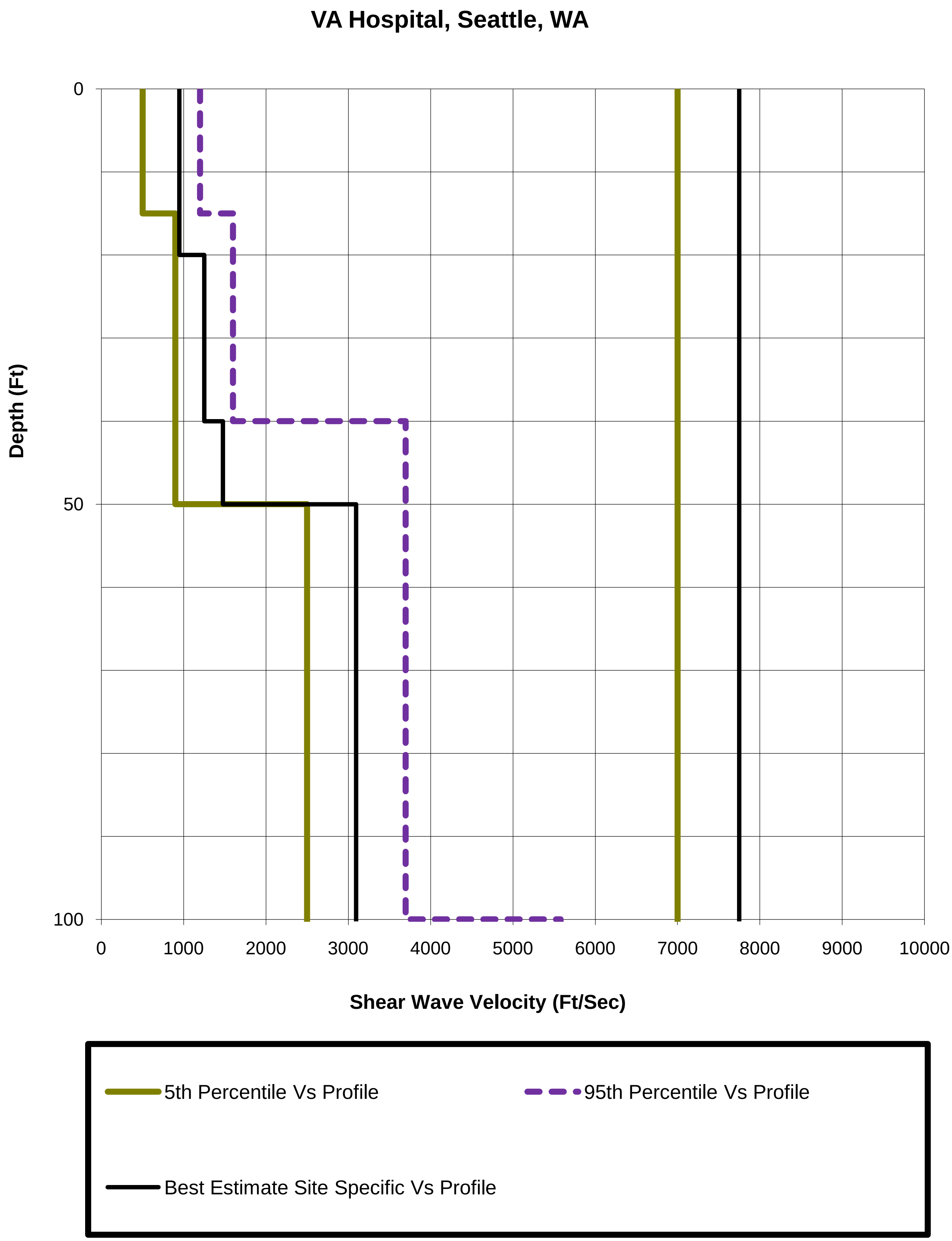
5% Damped Horizontal Response Spectra

Figure E-1

9851-006-00 10/28/2009 KHC:Site Specific Response Spectra.xls



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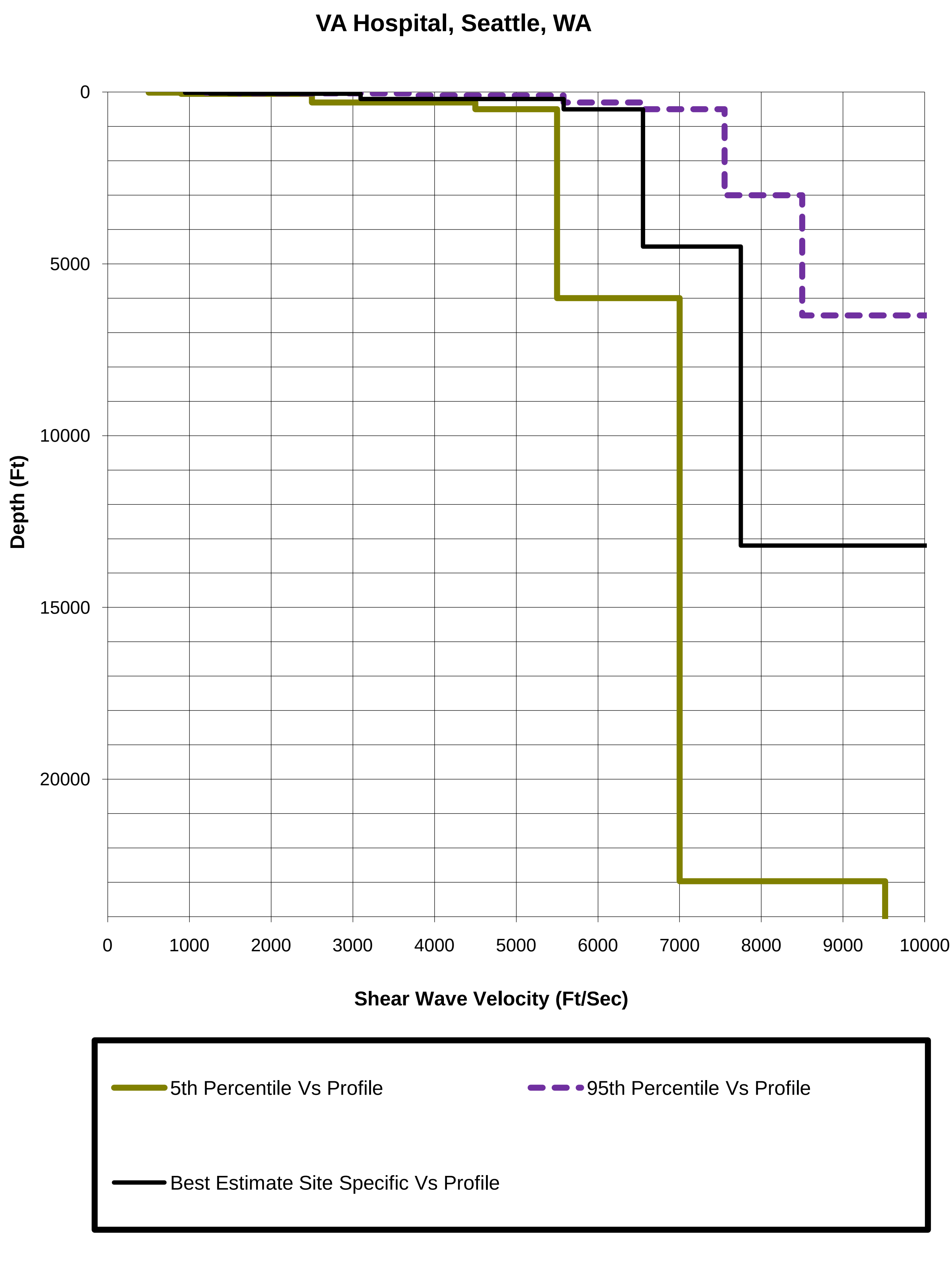
Near Surface Soil Shear Wave Velocity Profile

Figure E-2

9851-006-00 10/30/09 KHC: Vs Profile.xls



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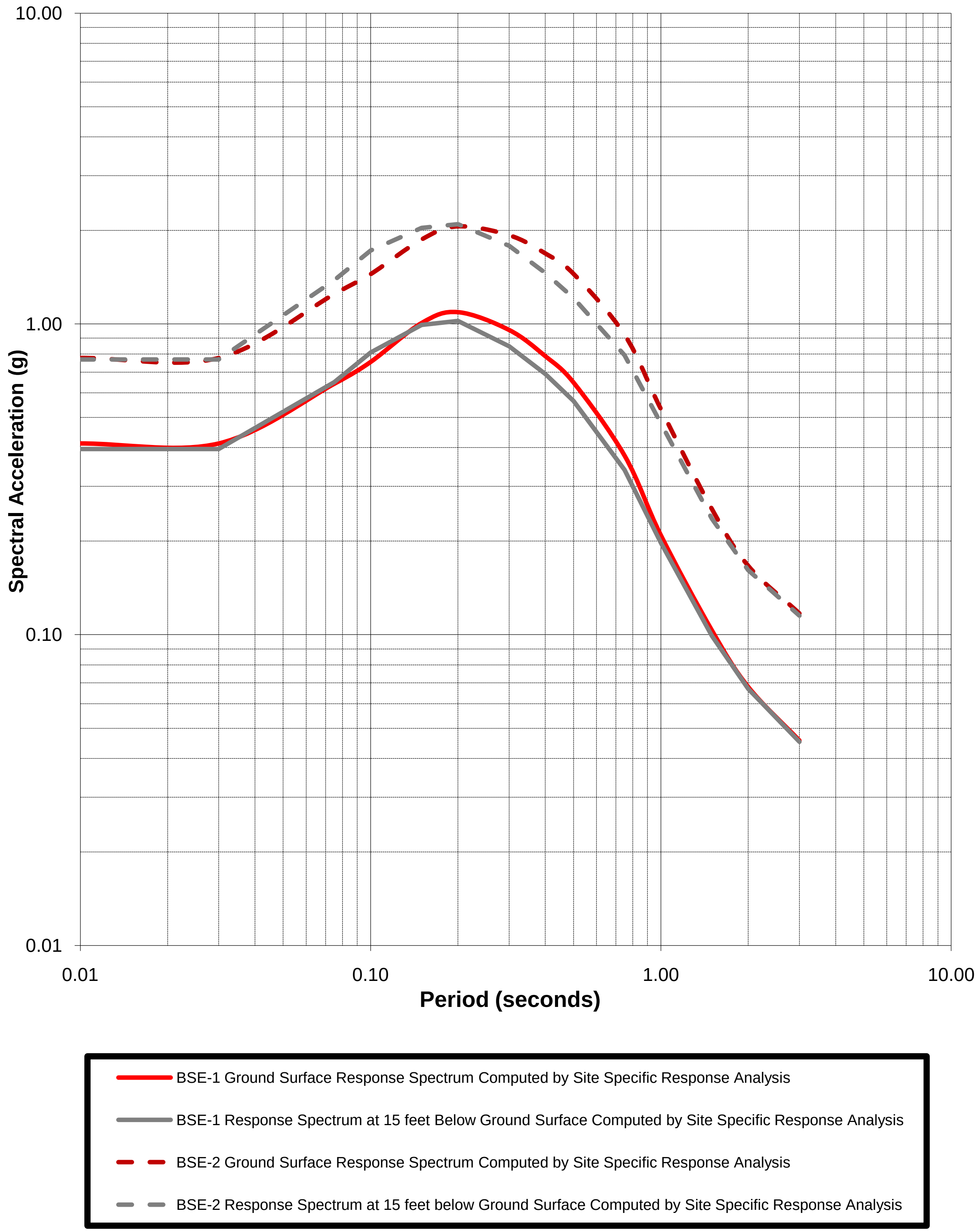
Soil Shear Wave Velocity Profile

Figure E-3

9851-006-00 10/30/09 KHC: Vs Profile.xls

VA Hospital, Seattle
Horizontal Response Spectra Computed by Site Specific Response Analysis



9851-006-00 10/28/2009 KHC:Site Specific Response Spectra.xls

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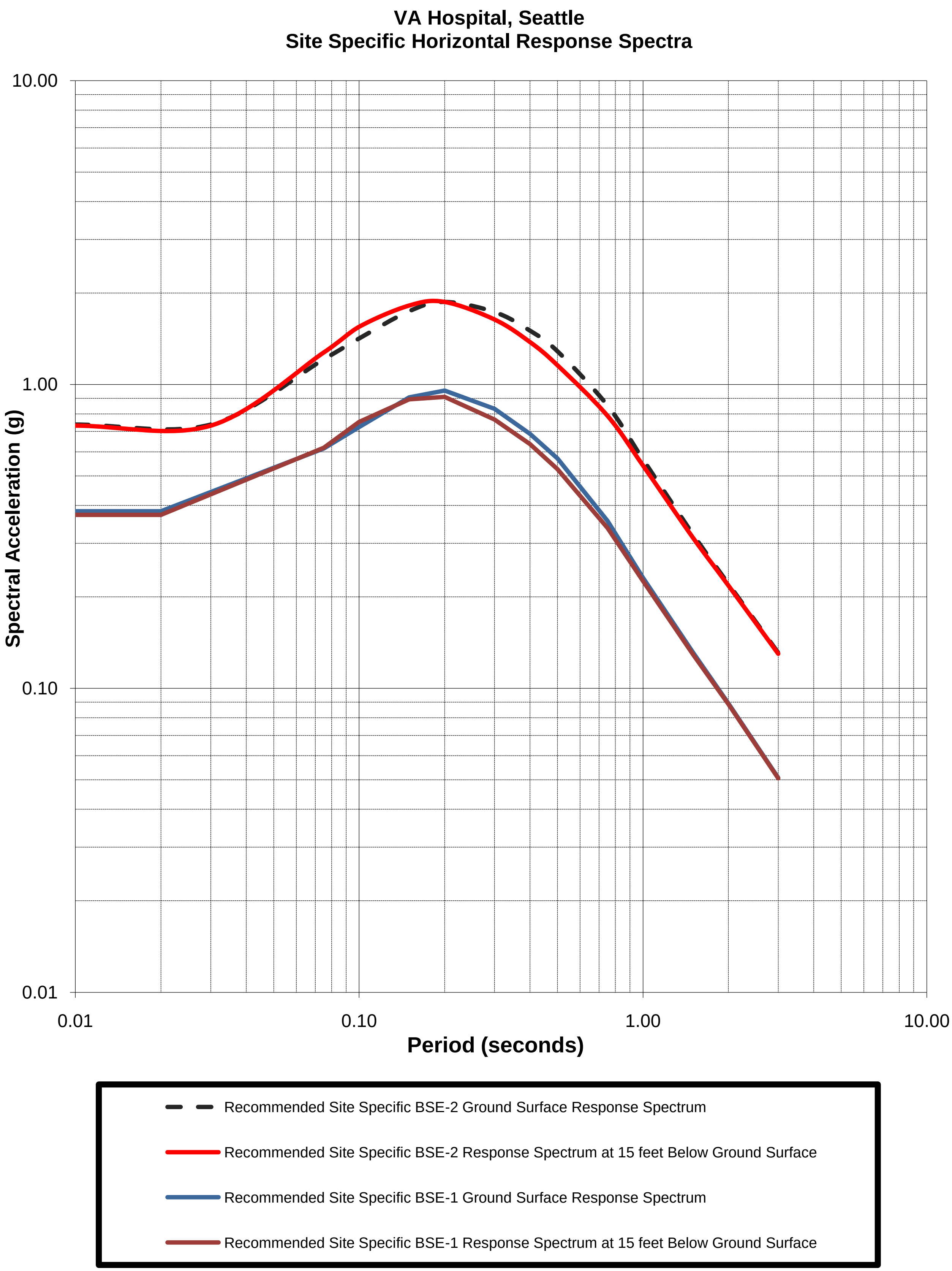
5% Damped Horizontal Response Spectra

Figure E-4



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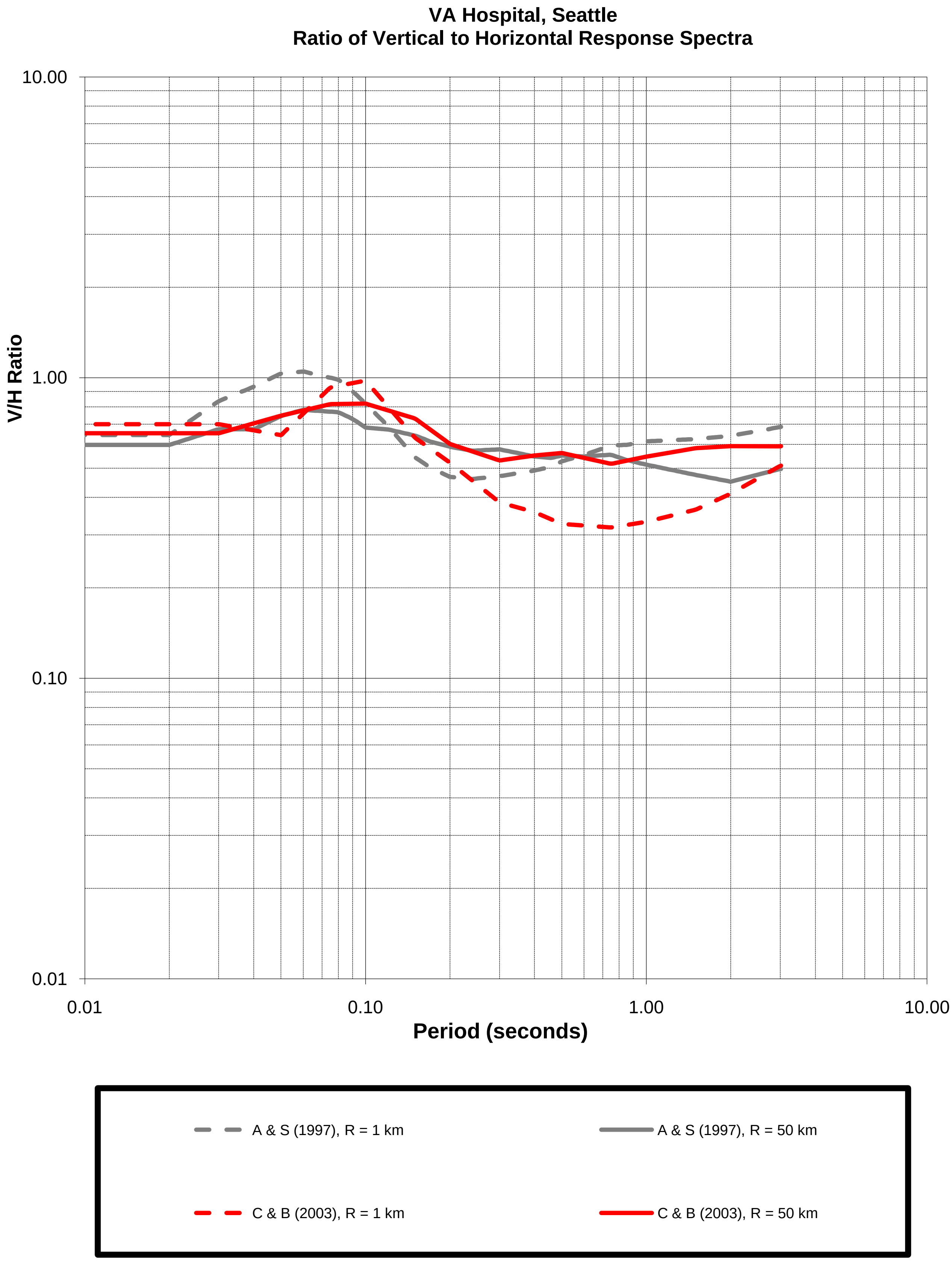
5% Damped Horizontal Response Spectra

Figure E-5



9851-006-00 10/28/2009 KHC:Site Specific Response Spectra.xls

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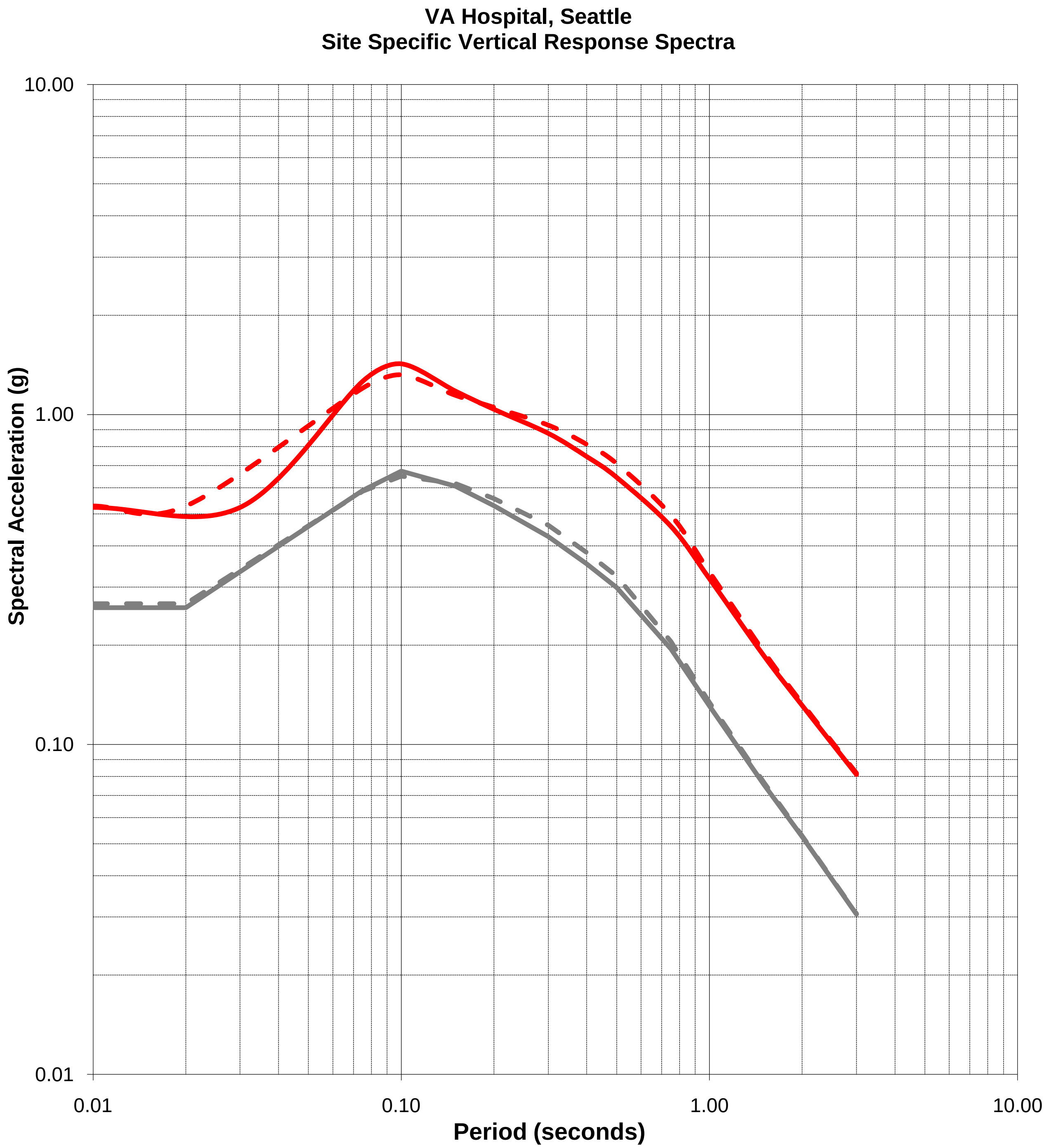
Ratios of Vertical to Horizontal Response Spectra

Figure E-6



Recommended Site Specific BSE-1 Ground Surface Vertical Response Spectrum

Recommended Site Specific BSE-1 Vertical Response Spectrum at 15 feet Below Ground Surface

Recommended Site Specific BSE-2 Ground Surface Vertical Response Spectrum

Recommended Site Specific BSE-2 Vertical Response Spectrum at 15 feet Below Ground Surface

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5% Damped Vertical Response Spectra

Figure E-7

9851-006-00 10/28/2009 KHC:Site Specific Response Spectra.xls

APPENDIX F
Ground Anchor Load Tests and Shoring Monitoring Program

APPENDIX F
GROUND ANCHOR LOAD TESTS AND SHORING MONITORING PROGRAM

Ground Anchor Load Testing

General

The locations of the load tests shall be approved by the Engineer and shall be representative of the field conditions. Load tests shall not be performed until the ground anchor grout and shotcrete wall facing, where present, have attained at least 50 percent of the specified 28-day compressive strengths.

Where temporary casing of the unbonded length of test ground anchors is provided, the casing shall be installed to prevent interaction between the bonded length of the ground anchor and the casing/testing apparatus.

The testing equipment shall include two dial gauges accurate to 0.001 inch, a dial gauge support, a calibrated jack and pressure gauge, a pump and the load test reaction frame. The dial gauge should be aligned within 5 degrees of the longitudinal ground anchor axis and shall be independently supported from the load frame/jack and the shoring wall. The hydraulic jack, pressure gauge and pump shall be used to apply and measure the test loads.

The jack and pressure gauge shall be calibrated by an independent testing laboratory as a unit. The pressure gauge shall be graduated in 100 pounds per square inch (psi) increments or less and shall have a range not exceeding twice the anticipated maximum pressure during testing unless approved by the Engineer. The ram travel of the jack shall be sufficient to enable the test to be performed without repositioning the jack.

The jack shall be independently supported and centered over the ground anchor so that the ground anchor does not carry the weight of the jack. The jack, bearing plates and stressing anchorage shall be aligned with the ground anchor. The initial position of the jack shall be such that repositioning of the jack is not necessary during the load test.

The reaction frame should be designed/sized such that excessive deflection of the test apparatus does not occur and that the testing apparatus does not need to be repositioned during the load test. If the reaction frame bears directly on the shoring wall facing, the reaction frame should be designed to not damage the facing.

Verification Tests

Prior to production ground anchor installation, at least two ground anchors for each soil type shall be tested to validate the design pullout value. All test ground anchors shall be installed by the same methods, personnel, material and equipment as the production anchors. Changes in methods, personnel, material or equipment may require additional verification testing as determined by the Engineer. At least two successful verification tests shall be performed for each installation method and each soil type. The ground anchors used for the verification tests may be used as production ground anchors if approved by the Engineer.

The allowable ground anchor load should not exceed 80 percent of the steel ultimate strength.

Ground anchor design test loads should be the design loads specified on the shoring drawings. Verification test tiebacks shall be incrementally loaded and unloaded in accordance with the following schedule:

Load	Hold Time	Load	Hold Time
Alignment Load (AL)	Until Stable	0.75DL	Until Stable
0.25 Design Load (DL)	Until Stable	1.0DL	Until Stable
AL	Until Stable	1.25DL	Until Stable
0.25DL	Until Stable	1.5DL	10 Minutes
0.5DL	Until Stable	AL	Until Stable
AL	Until Stable	0.25DL	Until Stable
0.25DL	Until Stable	0.5DL	Until Stable
0.5DL	Until Stable	0.75DL	Until Stable
0.75DL	Until Stable	1.0DL	Until Stable
AL	Until Stable	1.25DL	Until Stable
0.25DL	Until Stable	1.5DL	Until Stable
0.5DL	Until Stable	1.75DL	Until Stable
0.75DL	Until Stable	AL	Until Stable
1.0DL	Until Stable	0.25DL	Until Stable
AL	Until Stable	0.5DL	Until Stable
0.25DL	Until Stable	0.75DL	Until Stable
0.5DL	Until Stable	1.0DL	Until Stable
0.75DL	Until Stable	1.25DL	Until Stable
1.0DL	Until Stable	1.5DL	Until Stable
1.25DL	Until Stable	1.75DL	Until Stable
AL	Until Stable	2.0DL	10 Minutes
0.25DL	Until Stable	AL	Until Stable
0.5DL	Until Stable		

The alignment load shall be the minimum load required to align the testing apparatus and should not exceed 5 percent of the design load. The dial gauge should be zeroed after the alignment load is applied.

Proof Tests

The allowable ground anchor load should not exceed 80 percent of the steel ultimate strength.

Ground anchor design test loads should be the design loads specified on the shoring drawings. Proof test tiebacks shall be incrementally loaded and unloaded in accordance with the following schedule:

Load	Hold Time
AL	Until Stable
0.25DL	Until Stable
0.5DL	Until Stable
0.75DL	Until Stable
1.0DL	Until Stable
1.33DL	10 minutes
AL	Until Stable

The alignment load shall be the minimum load required to align the testing apparatus and should not exceed 5 percent of the design load. The dial gauge should be zeroed after the alignment load is applied. Depending upon the ground anchor deflection performance, the load hold period at 1.33DL (tiebacks) may be increased to 60 minutes. Ground anchor movement shall be recorded at 1, 2, 3, 5, 6 and 10 minutes during the load hold period. If the ground anchor deflection between 1 minute and 10 minutes is greater than 0.04 inches, the 1.33DL load shall be continued to be held for a total of 60 minutes and deflections recorded at 20, 30, 50 and 60 minutes.

Test Ground Anchor Acceptance

A test ground anchor shall be considered acceptable when:

1. For tieback verification tests, a tieback is considered acceptable if the creep rate is less than 0.04 inches per log cycle of time between 1 minute and 10 minutes, and the creep rate is linear or decreasing throughout the creep test load hold period.
2. For proof tests, a ground anchor is considered acceptable if the creep rate is less than 0.04 inches per log cycle of time between 1 minute and 10 minutes or less than 0.08 inches per log cycle of time between 6 minutes and 60 minutes, and the creep rate is linear or decreasing throughout the creep test load hold period.
3. The total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.
4. Pullout failure does not occur. Pullout failure is defined as the load at which continued attempts to increase the test load result in continued pullout of the test ground anchor.

Acceptable proof-test ground anchors may be incorporated as production ground anchors provided that the unbonded test length of the ground anchor hole has not collapsed and the test ground anchor length and bar size/number of strands are equal to or greater than the scheduled production ground anchor at the test location. Test ground anchors meeting these criteria shall be completed by grouting the unbonded length, as necessary. Maintenance of the temporary unbonded length for subsequent grouting is the contractor's responsibility.

The Engineer shall evaluate the verification test results. Ground anchor installation techniques that do not satisfy the ground anchor testing requirements shall be considered inadequate. In this case, the contractor shall propose alternative methods and install replacement verification test ground anchors.

The Engineer may require that the contractor replace or install additional production ground anchors in areas represented by inadequate proof tests.

Shoring Monitoring

Preconstruction Survey

A shoring monitoring program should be established to monitor the performance of the temporary shoring walls and to provide early detection of deflections that could potentially damage nearby improvements. We recommend that a preconstruction survey of adjacent improvements, such as streets, utilities and buildings, be performed prior to commencing construction. The preconstruction survey should include a video or photographic survey of the condition of existing improvements to establish the preconstruction condition, with special attention to existing cracks in streets or buildings.

Optical Survey

The shoring monitoring program should include an optical survey monitoring program. The recommended frequency of monitoring should vary as a function of the stage of construction, as presented in the following table.

Construction Stage	Monitoring Frequency
During excavation and until wall movements have stabilized	Twice weekly
During excavation if lateral wall movements exceed 1 inch and until wall movements have stabilized	Daily
After excavation is complete and wall movements have stabilized, and before the floors of the building reach the top of the excavation	Weekly

Monitoring should include vertical and horizontal survey measurements accurate to at least 0.01 feet. A baseline reading of the monitoring points should be completed prior to beginning excavation. The survey data should be provided to GeoEngineers for review within 24 hours.

For shoring walls, we recommend that optical survey points be established along the top of the shoring walls and at the curb line behind the shoring walls. The survey points along the top of the shoring wall should be spaced every other soldier pile for soldier pile walls. The points on the curb lines should be spaced approximately 25 feet apart. GeoEngineers recommends that a survey monitoring plan be developed for GeoEngineers' review prior to establishing the survey points in the field. If lateral wall movements are observed to be in excess of ½ inch between successive readings or if total wall movements exceed 1 inch, construction of the shoring walls should be stopped to determine the cause of the movement and to establish the type and extent of remedial measures required.

APPENDIX G
Report Limitations and Guidelines for Use

APPENDIX G
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Geotechnical Services Are Performed for Specific Purposes, Persons and Projects

This final report has been prepared for the exclusive use of the U.S. Department of Veterans Affairs and Stantec, Coughlin Porter Lundeen, Degenkolb Engineers, and their authorized agents. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with whom there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with the Client and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

A Geotechnical Engineering or Geologic Report Is Based on a Unique Set of Project-Specific Factors

This draft final report has been prepared for the New Mental Health Care Building/Seismic Retrofit of Nursing Tower/New Parking Garage project. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, do not rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

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[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org .

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;
- composition of the design team; or
- project ownership.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

Most Geotechnical and Geologic Findings Are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A Geotechnical Engineering or Geologic Report Could Be Subject To Misinterpretation

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

Contractors Are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor’s procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and to adjacent properties.

Read These Provisions Closely

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory “limitations” provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these “Report Limitations and Guidelines for Use” apply to your project or site.

Geotechnical, Geologic and Environmental Reports Should Not Be Interchanged

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings, or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants and no conclusions or inferences should be drawn regarding Biological Pollutants, as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and/or any of their byproducts.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.

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